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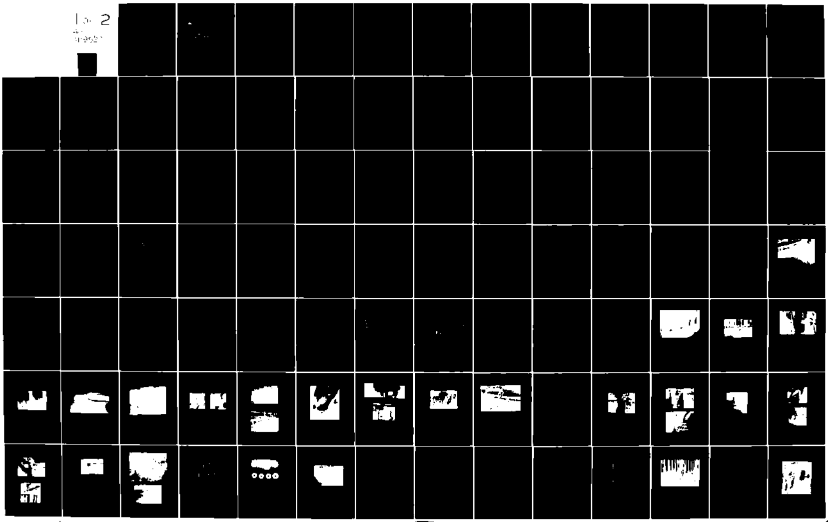

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Basic criteria for the design of the elements used in waterfront construction is presented for use by experienced engineers. The contents cover general requirements concerning piling, deck, and substructure framing, and hardware for waterfront structures, as well as specific criteria for various types of installations. A section on the strength evaluation of existing waterfront structures and a section on the deterioration of waterfront structures also are included.

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GENERAL CRITERIA FOR WATERFRONT CONSTRUCTION

DESIGN MANUAL 25.6

DEPARTMENT OF THE NAVY
NAVAL FACILITIES ENGINEERING COMMAND
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ABSTRACT

Basic criteria for the design of the elements used in waterfront construction is presented for use by experienced engineers. The contents cover general requirements concerning piling, deck, and substructure framing, and hardware for waterfront structures, as well as specific criteria for various types of installations. A section on the strength evaluation of existing waterfront structures and a section on the deterioration of waterfront structures also are included.

FOREWORD

This design manual is one of a series developed from an evaluation of facilities in the shore establishment, from surveys of the availability of new materials and construction methods, and from selection of the best design practices of the Naval Facilities Engineering Command, other Government agencies, and the private sector. This manual uses, to the maximum extent feasible, national professional society, association, and institute standards in accordance with NAVFACENGCOM policy. Deviations from these criteria should not be made without prior approval of NAVFACENGCOM Headquarters (Code 04).

Design cannot remain static any more than can the naval functions it serves or the technologies it uses. Accordingly, recommendations for improvement are encouraged from within the Navy and from the private sector and should be furnished to NAVFACENGCOM Headquarters (Code 04). As the design manuals are revised, they are being restructured. A chapter or a combination of chapters will be issued as a separate design manual for ready reference to specific criteria.

This publication is certified as an official publication of the Naval Facilities Engineering Command and has been reviewed and approved in accordance with the SECNAVINST 5600.16.

W. M. Zobel
Rear Admiral, CEC, U. S. Navy
Commander
Naval Facilities Engineering Command

WATERFRONT DESIGN MANUALS

DM No.	Superseded Chapters in Basic DM-25	Title
25.1	1 and 2	Piers and Wharves
25.2	3	Dockside Utilities for Ship Service
25.3	4	Cargo Handling Facilities
25.4	5	Seawalls, Bulkheads, and Quaywalls
25.5	6	Ferry Terminals and Small Craft Berthing Facilities
25.6	7	General Criteria for Waterfront Construction

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GENERAL CRITERIA FOR WATERFRONT CONSTRUCTION

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GENERAL CRITERIA FOR WATERFRONT CONSTRUCTION

Section 1. SCOPE AND RELATED CRITERIA

1. SCOPE. This manual contains general criteria for the design of piling, deck and substructure framing and bracing, and hardware and fittings for waterfront construction. Unless indicated otherwise, these criteria also apply to the design of offshore structures.

2. CANCELLATION. This publication, General Criteria for Waterfront Construction, NAVFAC DM-25.6, cancels and supersedes Chapter 7 of Waterfront Operational Facilities, NAVFAC DM-25, (October 1971), and any of the following changes which relate to Chapter 7: Change 1 dated February 1972, Change 3 dated July 1973, Change 5 dated March 1974, Change 6 dated August 1974, and Change 7 dated June 1975. Changes 2 and 4 are cancelled items.

3. RELATED CRITERIA. For related criteria, refer to the NAVFAC sources itemized below.

Subject	Source
Dockside Utilities for Ship Service...........	NAVFAC DM-25.2
Structural Engineering: Steel Structures..... Allowable stresses	NAVFAC DM-2.3
Structural Engineering: Concrete Structures.. Allowable stresses	NAVFAC DM-2.4
Structural Engineering: Timber Structures.... Allowable stresses	NAVFAC DM-2.5
Soil Mechanics, Foundations, and Earth Structures.................................. Pile capacity, size, length, and spacing; Interpretation of load tests on piles	NAVFAC DM-7
Fire Protection Engineering.................. Fire protection	NAVFAC DM-8

Section 2. PILING

1. GENERAL REQUIREMENTS.

a. Capacity.

(1) Capacity as a Structural Member.

(a) For pile sections embedded in the ground, refer to DM-7.

(b) For sections free standing in water, treat pile as a column having an unbraced length as shown in Figure 1. Where, due to long-term creep effects, the use of the coefficient of subgrade reaction would be inappropriate or if one is unavailable, or for conditions not covered by Figure 1, the following (conservative) assumptions may be made:

(i) In soft, cohesive soils, the point of fixity may be assumed to occur at a depth of 10 feet below the mud line for piles having an EI of 10×10^9 lb-in^2 or less and at a depth of 12 feet below the mud line for piles having an EI greater than 10×10^9 lb-in^2. (E = Modulus of Elasticity of Pile in pounds per square inch and I = Moment of Inertia of Pile in inches^4.)

(ii) In loose, granular soils and in medium cohesive soils, the point of fixity may be assumed to occur at a depth of 8 feet below the mud line for piles having an EI of 10×10^9 lb-in^2 or less, and at a depth of 10 feet below the mud line for piles having an EI greater than 10×10^9 lb-in^2.

(iii) For other cases, assume a point of fixity of a depth of 5 feet below the mud line.

The effective length factor K (See Figure 1) shall be taken as:

0.75 - when the deck structure is light, the piles have minimum embedment into the pile cap and there is no provision for moment transfer into the deck structure.

0.67 - when the deck structure is light and a provision is made for moment transfer by embedment or other device into the deck structure.

0.50 - when the deck structure is heavy and a positive means for moment transfer is provided.

These provisions do not apply if embedment is less than 10 feet into firm material or 20 feet into soft or loose material. If lesser penetration is provided, assume that the piles are hinged at their lower end. The indicated effective length factors apply if batter piles (minimum batter of 1 horizontal to 3 vertical) are provided to resist full lateral load, i.e., the plumb piles are not intended to resist lateral loads. If no batter piles are provided, increased K factors shall apply corresponding to condition where sidesway can occur.

(c) Design piles for a minimum eccentricity of 0.10 times the equivalent diameter of the pile. The moment resulting from this minimum eccentricity is not additive to the moments indicated by analysis of the applied loads.

(2) Capacity of the Ground to Support the Pile. See provisions of DM-7, concerning friction, end bearing resistance, and settlements of single piles and of pile groups.

(3) Lateral Load Capacity. For piles spaced more than three diameters center to center, assume that the soil reacts laterally on an equivalent pile having a diameter equal to three times the actual diameter of the pile. For closer spacing, reduce the assumed equivalent diameter proportional to the spacing. NOTE: A number of published design curves already include the effects of this spread of the load.

(4) Capacity of Existing Piles. See Section 6.

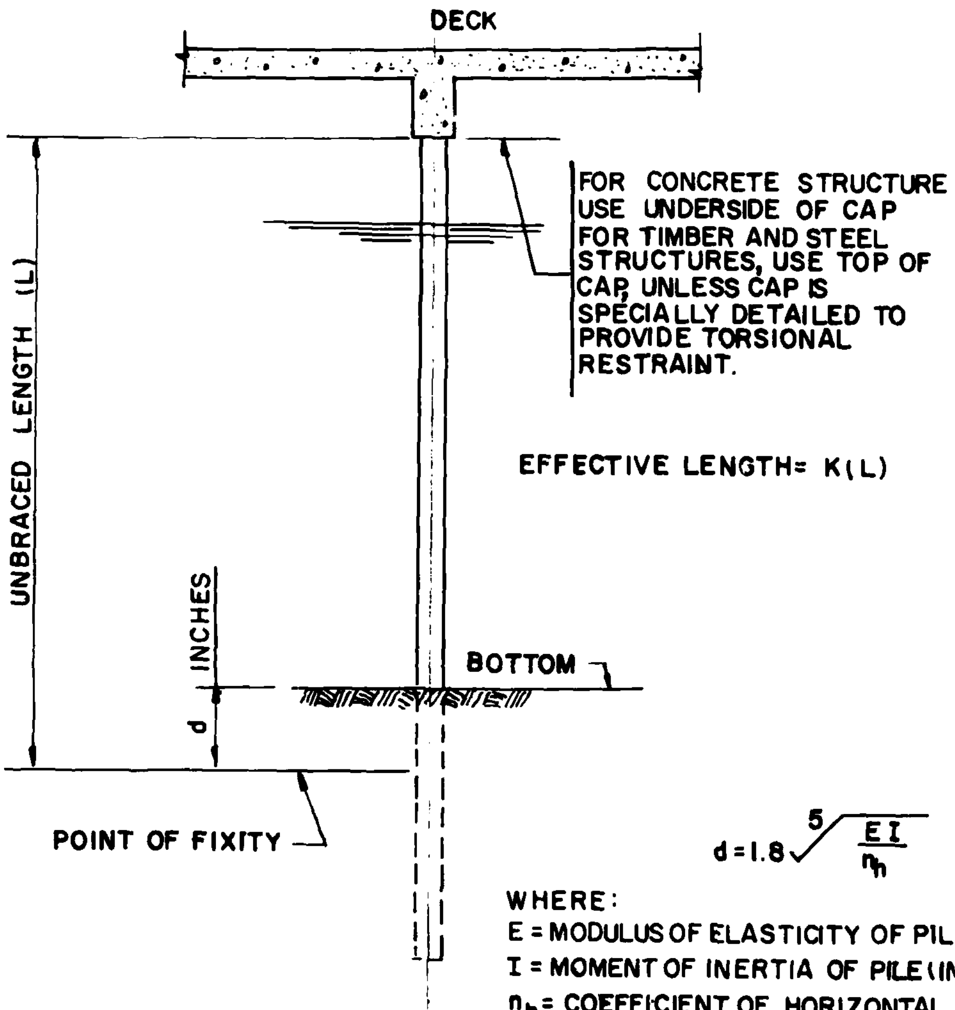
b. Details Applicable to All Pile Types.

(1) Minimum Penetration.

(a) Penetrate sufficiently into an acceptable bearing stratum to distribute the pile load within the supporting capacity of the soil. (See DM-7.)

(b) Penetrate sufficiently below probable future dredge depth to distribute the pile load within the supporting capacity of the soil. Discount resistance of soil which reasonable expectation indicates may be removed.

(c) Minimum values. No minimum values shall apply. However, if the pile penetration is less than 10 feet into firm material and less than 20 feet into soft or loose material, special provisions such as hardened tips driven into the refusing stratum, drilled sockets, or drilled dowels shall be made to secure the tips of the pile against lateral displacement due to inevitable eccentricities, lateral forces, and being out of plumb. Required lateral resistance shall be at least 5 percent of the design axial load. Effective length factors shall be increased as described above.



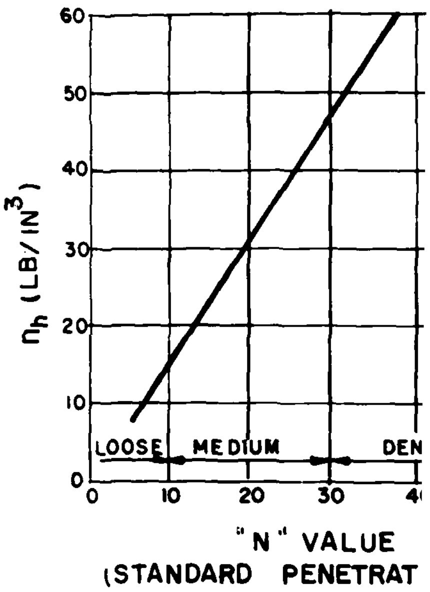
$$d = 1.8 \sqrt[5]{\frac{EI}{n_h}}$$

WHERE:

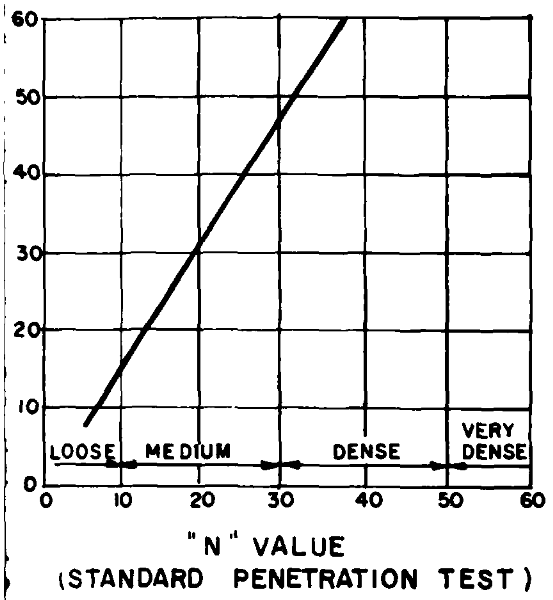
E = MODULUS OF ELASTICITY OF PILE (LB/IN^2)

I = MOMENT OF INERTIA OF PILE (IN^4)

n_h = COEFFICIENT OF HORIZONTAL SUBGRADE REACTIONS (LBS/IN^3)



USE THIS CHART TO
n_h FOR MEDIUM TO DE
SILT AND SAND SOILS



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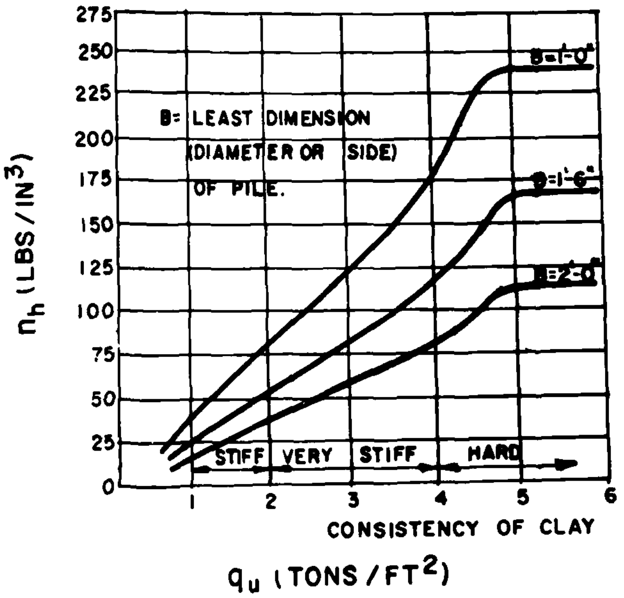


USE THIS CHART TO APPROXIMATE n_h FOR STIFT TO HARD CLAY SOILS.

FIGURE 1
Determination of
Unbraced Pile Length
25.6-4

(2) Tolerances on Installation. The following provisions relate to piling installed in open platforms where the piles project several feet or more above the mud line. For piles fully, or near fully, embedded in the ground, the provisions of NAVFAC Guide Specifications (TS Series) apply. (See references)

(a) Slope. Generally +4 percent from plumb or specified batter is a reasonable compromise between the needs of the design and the practicality of installation.

(b) Location of pile head. No limit provided that the structure can tolerate the revised pile spacing. However, residual stresses in the piles due to forcing the pile into the pile cap must be considered in evaluating the column capacity of the pile. No increase (or additional increase) in allowable stress should be applied to stress combinations which include these residual stresses. NOTE: For Kl/r (effective unbraced length, divided by radius of gyration) between about 40 and 100, these effects can be substantial. (See Section 6, paragraph 5.e.) But for fully embedded piles, Kl/r commonly is less than 40 and locked-in stresses can be neglected. Beware of the use of a driving frame which prevents lateral movements of the head of the pile and masks the existence of locked-in stresses. Ample edge distances shall be provided so that the piles will fit into the cap without excessive force or restraint. Allow for tolerance in the location of the pile head of at least 1.5 percent of the exposed height.

(3) Minimum Spacing.

(a) Provide for adequate distribution of the load on a pile group to the supporting soil.

(b) No minimum values are specified other than practical limitations to avoid piles interfering or intersecting with each other. A rule of thumb in occasional use is to use a center-to-center spacing equal to 5 percent of the pile length.

(4) Pile Caps in Contact with the Ground. Piles shall be designed to carry the entire superimposed load with no allowance made for the supporting value of the material between the piles.

(5) Connection of Piles to Caps.

(a) No Tension in Piles - Timber Caps. Tops of piles shall be secured to caps with spiral drive drift bolts, metal straps, or scabs.

(b) No Tension in Piles - Concrete Caps. Tops of piles shall have a minimum 4-inch embedment.

(c) No Tension in Piles - Steel "H" Piling in Concrete Caps. (See requirements in paragraph 2.f.(3).)

(d) Tension in Piles - Timber piles in Concrete Caps. Shoulder and embed butts to satisfy requirements of shear stress in the timber and diagonal tension stress in the concrete. Timber connectors are permitted as an alternate method of stress transfer from cap to piles.

(e) Tension in Piles - Concrete Piles in Concrete Caps. Dowel into cap.

(f) Tension in Piles - Steel Piles in Concrete Caps. Calculate bond resistance as 0.02 f'_c for contact surfaces which are cast "in the dry" and as 10 psi for contact surfaces which are cast below water (tremie).

(g) Tension in Piles - Timber Piles with Timber Cap. Provide scabs and shear bolts or provide metal straps.

(6) Batter Piles. Connections to adjacent piles in group shall be capable of developing the calculated tension, but not less than a tension equal to one-half the compression load less the dead weight in the pile.

(7) Splicing. Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the pile during installation and subsequent thereto. Splices shall be of adequate strength to transmit the vertical and lateral loads (including tensions), and the moments occurring in the pile section at the location of the splice, without exceeding the allowable stresses for such materials as established in the DM-2 Series for Service Classification B. Except for piles which can be visually inspected after driving, splices shall develop at least 50 percent of the capacity of the pile in bending or the moment and shear that would result from an assumed eccentricity of the pile load of 3 inches, whichever is the greater requirement. For pile splices which can be inspected and reinforced after driving, no minimum strength requirement is specified.

(8) Mixed Types or Capacities of Piling and Multiple Types of Installation Equipment or Methods. Use of such is permitted, provided that analysis is made of the effects on the superstructure of differential elastic shortening and settlement. Consider conducting load tests to evaluate differential settlements and spring values for piles.

(9) Slope of Batter Piles. Unless special provisions are made for the difficulties of installation and the effects of diminution of the hammer blow on the capacity, keep the slope of the batter piles to 1 horizontal to 2 vertical or steeper, and preferably 1 horizontal to 2.5 vertical.

2. REQUIREMENTS FOR SPECIFIC TYPE OF PILES.

a. Untreated Timber Piles.

(1) Piles shall conform to ASTM D25. (See References.)

(2) Cut-off. Cut-off at or below permanent ground water level. In areas having semidiurnal tides, cut-off shall be at or below a level equal to R/3 above MLW, where R is the tidal range. In areas having a diurnal tide, cut-off shall be at MLW.

(3) Borers. Untreated piles shall not be used in locations where they will be exposed to borers, except that use of untreated fender piles will be permitted where experience demonstrates that such use is justified. In general, untreated piles should not be used where they will be free-standing in salt or brackish water.

(4) Seasoning. Not required.

(5) Protection for Tops of Piling. Not required.

(6) Hardware and Fittings. (See Section 4.)

(7) Species. Any species of wood may be used that will provide the necessary structural capacity and that will withstand the driving stresses.

(8) Peeling. Not required.

b. Treated Timber Piles.

(1) Piles shall conform to ASTM D25. (See References.)

(2) Preservative Treatment. Treated marine piling shall bear the appropriate American Wood Preservers Bureau (AWPB) Quality Mark as follows: MP-1 (dual treatment) for use in areas of extreme borer hazard and in marine waters where limnoria and pholad attack may be expected or where oil slicks may contribute to borer attack, and MP-2 for other conditions where pholad attack is not expected, MP-4 treatment (water-borne perservatives) may be considered. (See References.) For specific requirements at particular locations, consult NAVFAC field division, Applied Biology Office. (See Section 5, paragraph 9. for properties of treated wood.)

(3) Seasoning. Required prior to treatment.

(4) Species. Preferably southern pine or douglas fir. Use of other species may be made subject to NAVFAC approval. AWPB Quality Control Standards include a requirement that the species be southern pine or douglas fir. It is not normally necessary to specify species separately. In areas where treatable soft woods are scarce, and if a treated pile is required, consider the use of concrete piling.

(5) Protection for Pile Tops. Cut ends preferably shall be treated by puddling creosote. Puddling is accomplished by using a sheet metal ring to form a reservoir on top of the pile. The reservoir is filled with creosote oil and left to stand for 8 to 12 hours. Alternate protection methods include coating pile tops with pitch (with or without sheet metal covers). NOTE: Use of sheet metal covers as end protection for fender piles is discouraged. The covers are easily torn by impact and become a personnel hazard. However, sheet metal covers for bearing piles under cross caps provide good protection. In general, piling to be covered with other structural members should be fitted with waterproof caps.

c. Untreated and Treated Timber Piles.

(1) Limitations on Use. Timber piles installed to end-bearing on rock, hardpan, caliche, or other semi-cemented materials require special care in installation to prevent damage.

(2) Lagged Piles. Double lagging shall be connected to the basic pile material to transfer the full pile load from the basic pile material to the lagging. The connection for single lagging shall be proportioned for half the pile load.

d. Precast (Including Prestressed) Concrete Piles.

(1) Minimum Dimensions. 12 inches for piles of uniform section and 8 inches for tapered piles.

(2) Cover. Minimum clear cover for reinforcement for permanent installations in salt water shall be 3 inches. For temporary installations and in fresh water, cover requirements shall conform to the requirements of DM 2.4 for normal exposure conditions.

(3) Minimum Reinforcement. Excluding prestressed piles, minimum longitudinal reinforcement shall be 1.5 percent of the total cross section.

(4) Ties. Provide spirals or ties for longitudinal reinforcement. Proportion spirals and ties in accordance with the ACI provisions for structural columns except provide additional ties or spirals at ends as indicated in Definitive Designs for Naval Shore Facilities, NAVFAC P-272, Drawing No. 1293323.

(5) Impact. Forces induced by handling and driving shall be used with a load factor of 1.25 (allowable overstress of 33 percent).

(6) Jetting. Where jetting is contemplated, the jet pipe should be cast into the pile.

(7) Class of Concrete. Use 5000 psi minimum for prestressed concrete. Use 4000 psi minimum for non-prestressed concrete.

(8) Standard Details. (See NAVFAC P-272.)

(9) Minimum Residual Prestress. 700 psi.

(10) Minimum Wall Thickness (Piles with Voids). Provide minimum of 1-1/2 inch clear cover on inside face (surface of void). In no case should wall thickness be less than 4 inches.

(11) Venting. If a void is provided which extends through to the lower end of the pile, vent the pile head to prevent the build up of internal hydraulic pressure during driving.

(12) Tolerances. Voids, when used, shall be located within 3/8-inch of the position shown on the plans. The maximum departure of the pile axis from a straight line, measured while the pile is not subject to bending forces, shall not exceed 1/8-inch in any 10 foot length or 3/8-inch in any 40 foot length. Overall sweep shall not exceed 0.1 percent of the pile length.

e. <u>Cast-in-Place Concrete Piles</u>.

(1) General. This is not a good type of pile for exposed locations and long unbraced lengths, especially when exposed to salt water corrosion.

(2) Casings.

(a) Casings shall have adequate strength to withstand the driving stresses and resist the distortion due to driving adjacent piles.

(b) Except for portions of the pile embedded more than 5 foot below the ground level, leave the casings in place permanently. However, consider the portion of the casing above this level as sacrificial metal when estimating the structural capacity unless protective coatings or corrosion-resistant alloys are used. Metal of 1/8-inch or less in thickness shall not, in any case, be considered as contributing to structural capacity.

(3) Minimum Tip Diameter. 8 inches.

(4) Reinforcement. Sections of piling which are above the point of fixity (as specified in paragraph 1.a.(1)) shall have sufficient casing thickness to provide residual metal at termination of design service life equal to minimum required reinforcement. Otherwise, such sections shall be reinforced with lateral ties and longitudinal bars in the same manner as precast piles. Cover requirements shall be as for precast piles. Spacers shall be provided for longitudinal reinforcement to assure that cover requirements are maintained.

(5) Class of Concrete. Salt water - use 4,000 psi minimum. Fresh water - use 3,500 psi minimum.

f. Steel "H" Piles.

(1) Minimum Thickness of Metal. Thickness of metal shall be determined from consideration of loss of section as established in DM-2.3, unless corrosion protection is provided as described in paragraph (2) below. The minimum thickness shall not be less than 0.40 inches. Splice plates shall not be less than 3/8 inches thick.

(2) Corrosion Protection. When the required minimum thickness of metal is excessive, corrosion protection in the form of concrete, bituminous or plastic (epoxy) coatings, or cathodic protection shall be provided. When coatings are utilized, exercise special care to avoid damaging coatings during driving or apply coatings after driving the piles. Bituminous or plastic coatings shall not be considered effective below the mud line and they require special care so they are not damaged through rubbing against the driving frame or template. In tropical environments, and other locations where corrosion is particularly severe, encase piling with concrete to 2 feet below MLLW.

(3) Cap Plates. Cap plates are not required for steel piles embedded in a concrete pile cap. Where structural design depends on bending in the piles for stability, tie the tops of steel piles into the cap with reinforcing rods or structural sections welded to the pile and lapping the cap reinforcement.

(4) Lugs, Scabs and Core-stoppers. Lugs, scabs, and core-stoppers may be used to increase the capacity of end-bearing steel piles. In fact, where "H" sections are used as friction piles and where such piles are subject to severe impact loadings, it is desirable to fit the ends with such devices to prevent "driving" the piles under the action of the impacting load.

(5) Hardware and Fittings. (See Section 4.)

(6) Limitation on Use. The tips of all steel "H" piles having a thickness of metal less than 0.5 inches, which are driven to end bearing on sound rock by an impact hammer, shall be reinforced. Observations of penetration resistance and the operation of the equipment shall be conducted so as to terminate driving directly when the pile reaches refusal on the rock surface.

g. Bell and Cylinder Piling and Screw Cylinders.

(1) Usage. Economical use of cylinder piles is normally attained where heavy moving loads must be supported, or where the piles must bear on rock or other hard bottom, at shallow depth, without sufficient overburden for lateral support.

(2) On Hard Bottom (rock or hardpan). In general, use a constant diameter shaft without the bell. Sloping surfaces should be leveled to receive the shaft. The need for anchorage in the form of dowels or keys should be considered.

(3) On Other Bottom Surfaces. Consider shaft and bell construction if loads are heavy enough to warrant use of the bell. Except where supported on piling, embed the bell 2 to 4 feet into firm material.

(4) Minimum Reinforcement. None required.

(5) Embedment of Piling Into the Bell. As required for transfer of load. Where tremie placement is employed, use 10 psi bond resistance between pile and concrete, plus compression resistance of top of pile bearing in bell.

(6) Protection for Reinforcement. Same as concrete piling, except reduce to 1-1/2 inches where permanent outer shell is provided.

(7) Thickness of Metal Shell and Corrosion Protection. Same as for "H" piling.

(8) Installation.

(a) Tremie placement of concrete fill is permitted.

(b) Provide for final cleanout of the bell or base of the cylinder immediately before concreting.

h. Steel Pipe Piles. Pipe piles shall conform to the applicable requirements for both steel "H" piles and cast-in-place concrete piles, except as modified below.

(1) Material. ASTM A252, unless otherwise approved.

(2) Open-end piles.

(a) Pipes installed open end shall be reseated to full bearing after cleaning. If the pipe shell shows 2 inches or more of penetration on reseating, reclean and redrive in successive cycles until penetration on redriving is less than 2 inches.

(b) If the leakage of water into the pile is minor, the pile shall be pumped out and 1 cubic yard of grout shall be placed before the balance of the concrete is installed. If the leakage of water makes it inadvisable to attempt to place concrete in the dry, then the shell shall be filled to its top with clean water. The concrete is placed by the tremie method to the top of

the pipe in one continuous operation or by using a grout seal of the same strength as the specified concrete. The grout seal, if used, shall be deposited by means of a grout pipe to an elevation of at least 3 feet above the bottom of the pile. After a sufficient time has elapsed to allow the grout to set, the pile shall be pumped dry and the remaining space filled with concrete.

(3) End Closure. For friction piles, end closures shall not project more than 1/2 in. beyond the outer limits of the pile shell.

i. Composite Piles. Size, capacity, and details of each section shall conform to the applicable provisions stated above.

j. Sheet Piling - Steel. Requirements are the same as previously established for steel "H" piles, "H" piling, except as modified below.

(1) Splices. Splices shall consist of full penetration butt welds. Splices where the upper section of sheet is driven as a follower to the lower section without positive connection between the two sections will not be permitted even though splice may occur at a point of zero moment.

(2) Connection to Cap. For concrete caps, use 6 in. min. embedment. The use of rods or structural sections for anchorage of sheet piling into the cap is not required. For steel channel caps, tack weld each sheet to the cap.

(3) Sleeves and Openings. All sleeves and openings for utilities passing through the sheets shall be detailed to prevent loss of fill. (See Figure 2.)

(4) Minimum Thickness of Metal. For exposed face of cofferdams, use 1/2 inch minimum. Elsewhere, provide thickness consistent with design service life. Thickness less than 3/8-inch may be used, but must be justified.

k. Sheet Piling - Concrete. Piling shall conform to the requirements stated above for precast concrete piles except as modified below.

(1) Joints. Joints shall be flushed and grouted and shall be tight to the mud line. Use of plastic sleeve is recommended. (See Figure 3.)

(2) Ties. Spirals or ties for longitudinal reinforcement are not required, except at tip and driving ends.

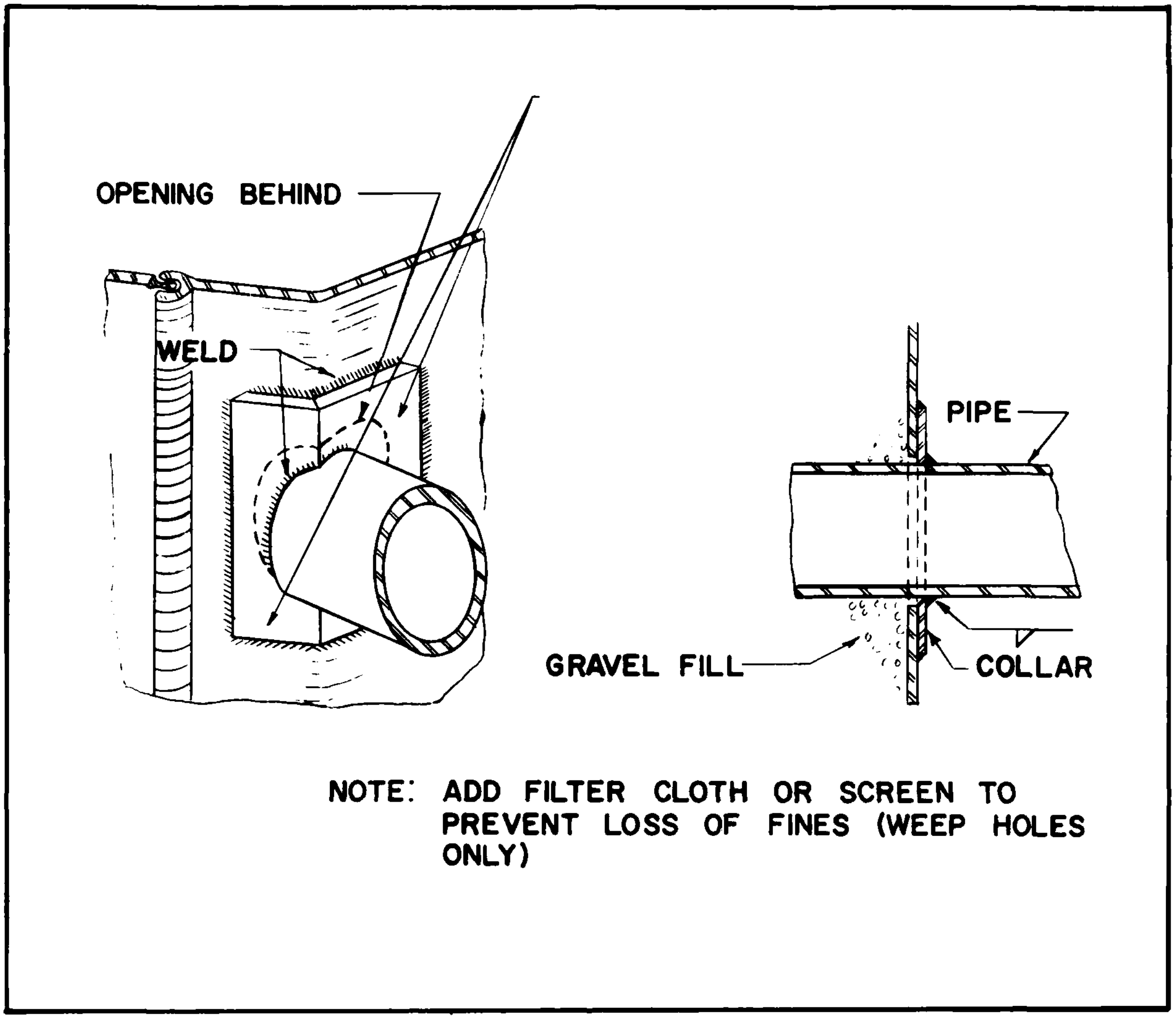


FIGURE 2
Sleeve Details

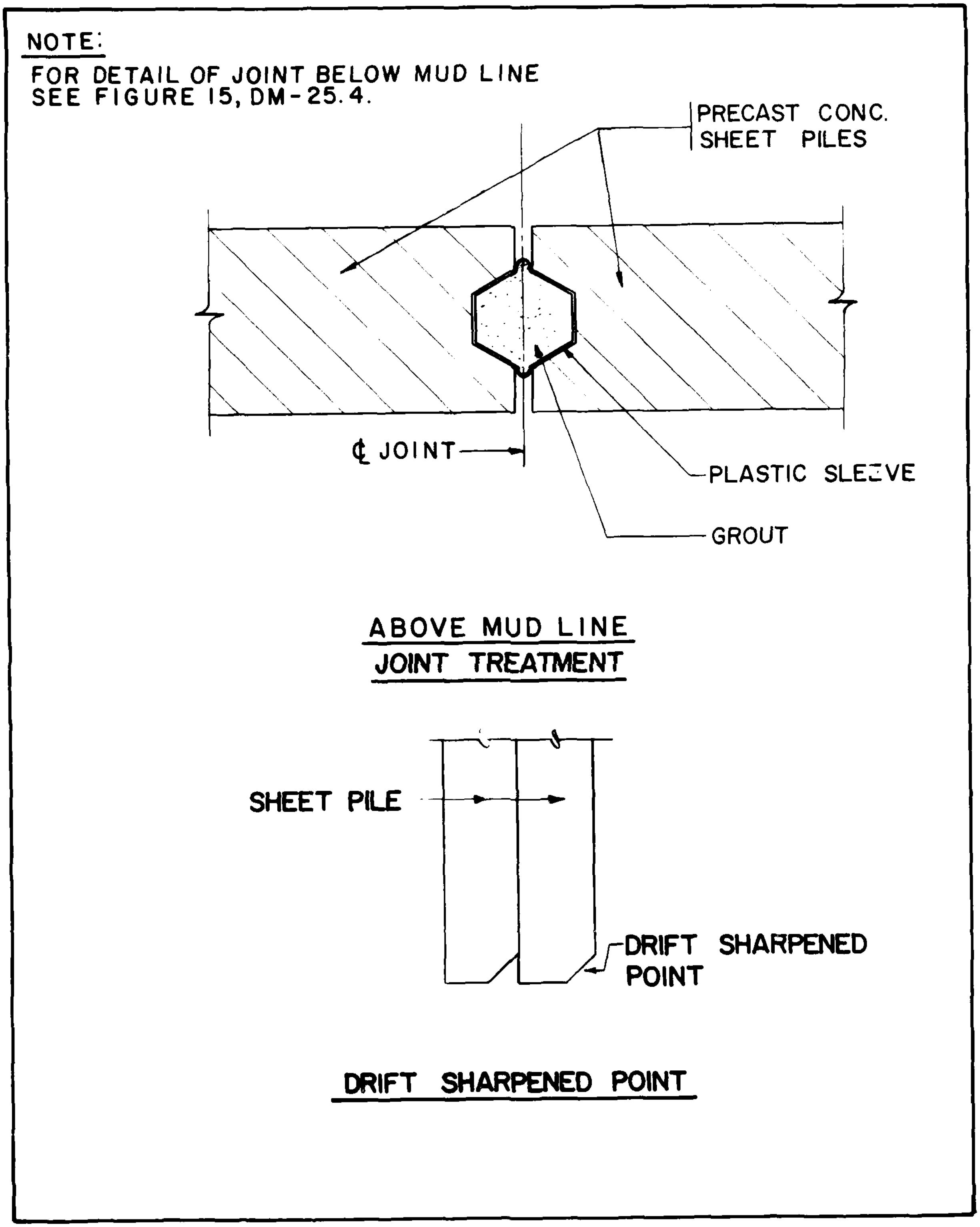


FIGURE 3
Concrete Sheet Piling Details

(3) End of Sheets. Cast sheets with a drift sharpened point. (See Figure 3.) Embed tops of sheets 6 inches into a continuous cap.

(4) Sleeves and Openings. Similar to steel sheet piling.

l. Sheet Piling - Timber. Piling shall conform to the requirements stated above for treated and untreated timber piles except as modified below.

(1) Treatment. Timber sheet piling shall bear the AWPB Quality Mark MLP. Types of treatment shall be as described for treated timber piles.

(2) Joints. Joints shall be tongue and groove, or splined (or Wakefield Type sheeting may be used). Sheet piling shall be tight to the mud line. Tongue and groove sheets and splines shall have a loose fit.

(3) Drift Sharpening. Drift sharpen sheets.

(4) Tops of Sheets. Tops of sheets shall be drift bolted or spiked to a continuous timber cap. Width of the cap shall be equal to or greater than the thickness of the sheet piling and thickness of the cap shall not be less than 2 in. Where a concrete cap is used, embed sheets 6 in. into the cap.

(5) Sleeves and Openings. Similar to steel sheet piling.

Section 3. DECK AND SUBSTRUCTURE FRAMING AND BRACING

1. SUBSTRUCTURE. The "substructure" shall include pile caps, under deck bracking and other structural members (other than stringers) at and below the level of the pile caps.

a. Piles Caps - All Types. The effects of differential settlements of piles shall be investigated for the following conditions:

(1) Where heavy concentrated loads occur.

(2) Where piles are long.

(3) Where there is an appreciable variation in pile lengths.

(4) Where pile types or methods of installation vary.

Differential settlement will not appreciably affect ultimate strength if yield of the cap can occur without buckling or fracture.

b. Timber.

(1) Hardware and Fittings. (See Section 4.)

(2) Species and Preservative Treatment. Except for temporary structures, substructure timbers shall be given a preservative treatment and shall be a species which will accept deep treatment such as southern pine or douglas fir. For permanent structures, use of untreated timber presumed to be of superior resistance to borers and decay is discouraged. Experience suggests that such resistance is inadequate for long-term use and is unreliable. Species which do not accept perservative treatment shall be encased, or shall be used with full awareness of the need for maintenance.

(3) Seasoning. Use only seasoned timber for framing.

(4) Minimum Dimension. Use minimum 3 in. (nominal size) in and below splash zone. Above splash zone, minimum size shall be 2 in. (nominal).

(5) Retention and Penetration of Preservative. Conform to requirements of AWPB Standard MLP. Select type of treatment as described for treated timber piles.

c. Concrete.

(1) Cover. Conform to requirements of Section 2, paragraph 2.d.(2).

(2) Chamfer. Chamfer all corners 3/4 inch minimum.

(3) Class of Concrete.

(a) Precast concrete and concrete in a salt water environment, use 4000 psi minimum.

(b) Other types and applications, use 3000 psi minimum.

(4) Dimensional Changes. Concrete undergoes dimensional changes due to temperature and shrinkage and tends to swell if cast in-the-dry and then immersed in water. If such changes are prevented, stresses develop in the concrete. To minimize such stresses, the following construction details should be observed:

(a) Where thin concrete sections abut massive sections, allow for differential movements or break contact length into short segments by expansion joints.

(b) Where new concrete abuts old, allow for differential movements, or break contact length into short segments by expansion joints.

d. Steel. Conform to the requirements for steel "H" piles.

2. DECK. The "deck" shall include treads, planks, slabs, stringers, and other elements supported by the pile caps.

a. Timber. Timber used in the deck structure shall conform to the requirements for substructure framing and bracing except as modified below.

(1) Treatment. Except for conditions described in paragraphs 1.(a) and (b) below, deck framing and bracing shall be given a preservative treatment. Do not use creosote treatment on walking surfaces or surfaces which normally will be touched by people (handrails, for example). Treatment is not required for the following:

(a) Temporary Structures.

(b) Timbers above mean high water level, provided that resistant species (generally hardwoods) are used and the construction is detailed to provide for circulation of air around the timber and to minimize the extent of faying surfaces.

(2) Hardware and Fittings. (See Section 4.)

(3) Treads.

(a) Preferably oak, maple, black gum, or other species resistant to wear.

(b) Not over 12 inches wide.

(c) Not less than 3 inches thick (nominal size).

(d) Provide minimum 3/8 inches clear between treads.

(e) Preferably attach treads to planks with drive screws. Where nailed, use minimum 20 penny nails.

(f) Do not consider treads as load-carrying members, but they can be considered in evaluating distribution of load to the planks.

(4) Planks.

(a) Not over 12 inches wide.

(b) Provide minimum 3/8 inches clear between planks.

(c) Preferably attach planks to stringers or nailers with drive screws. Where nailed, use minimum 20 penny nails.

(5) Details.

(a) Provide solid bridging between stringers at points of bearing and at intermediate lines at 20 foot maximum spacing.

(b) Stringers to bear full width on caps. Lap adjacent stringers over caps.

b. Concrete.

(1) General. Conform to requirements for substructure framing and bracing except that cover for reinforcement shall conform to requirements of DM-2.4 for normal exposure conditions.

(2) Deck Finish. Broom finish to provide skid-resistant surface.

c. Steel. Conform to the requirements for substructure framing and bracing.

d. General Provisions for All Types of Materials.

(1) Safety Provisions.

(a) Provide curb logs to prevent vehicles driving off deck. Curbs should be a minimum 10 inches high (preferably 12 inches high) by whatever width is required for strength. Provide scuppers as required. Minimum size of scuppers to be 2 inches by 8 inches.

(b) For non-berthing faces, provide railing (fixed or removable) for safety of personnel.

(2) Drainage.

(a) Pitch deck slab a minimum 1/16 inch per foot to drain to scuppers or collection points.

(b) Where scuppers are permissible and feasible, utilize in preference to drain holes. Size as stated above.

(c) Where drain holes are required, size as required for local rainfall intensity (25 year storm) with minimum 4 inch diameter. Locate drain holes between each pair of pile bents and at 20 to 30 foot spacing parallel to pile bents.

(d) Provide additional drain holes in service pits and in recesses for mooring fittings.

(e) Provide 1-1/2 inch diameter drains in rail slots, two between each pair of pile bents.

(3) Special Drainage for Petroleum Offloading and for Fueling Piers.

(a) Generally, an intercept system is required to collect oil spills. In normal operation, deck drainage outfalls through the sump pumps into the harbor. If an oil spill occurs, pressing a deck-mounted button closes a motor-operated outfall valve and starts the sump pumps which pump the spill to a collection point. When the spill drainage procedure is completed and all oil is removed from the system, the system reverts to normal operation.

(b) In some cases, consideration of contamination of rainwater runoff due to contact with residual drippings on the deck requires collection of all deck drainage.

Section 4. HARDWARE AND FITTINGS (PERMANENT INSTALLATIONS)

1. SALT WATER - IN OR BELOW SPLASH ZONE.

a. Minimum Diameter of Bolts. 1-inch.

b. Minimum Thickness of Metal in Straps and Fittings. 1/2 inch.

c. Coatings. All hardware and fittings shall be galvanized except that galvanizing of ogee washers shall be optional.

d. Washers. Provide bolts with ogee washers. Plate washers shall not be used without special approval of NAVFAC. In general, not more than two washers should be allowed under any bolt head or nut. Inclined bolts should be fitted with beveled washers.

e. Size of Bolt Holes. All bolt holes in timber (other than holes for drift bolts) are to be drilled with a bit having a diameter 1/16 inch larger than the diameter of the bolt shank. Alignment of bolt holes shall allow insertion by tapping with a mallet. Driving or force fitting of bolts is not allowed. Holes for drift bolts to be 1/8 inch less in diameter than the bolt diameter. All drill bits shall be kept sharp and feed rate controlled to produce shavings, rather than chips.

f. Locking of Bolts. Check the nuts, weld, or provide other positive means of locking the bolts.

2. SALT WATER - ABOVE SPLASH ZONE. Requirements shall be as for installations in or below splash zone except as modified below:

a. Minimum Diameter of Bolts. 3/4 inch.

b. Minimum Thickness of Metal in Straps and Fittings. 3/8 inch.

c. Washers. Use of ogee or plate washers shall be optional. Minimum thickness of plate washers shall be 1/4 inch.

3. FRESH WATER. Requirements are the same for installation in salt water, in or below splash zone, except as modifed below:

a. Minimum Diameter of Bolts. 5/8 inch.

b. Minimum Thickness of Metal in Straps and Fittings. 1/4 inch. (3/8 inch is preferable).

c. Washers. Use of ogee or plate washers is optional. Minimum thickness of plate washers is 1/4 inch.

4. SPECIAL APPLICATIONS.

a. Stainless Steel Fittings. May be useful for special applications, if cost is warranted.

b. Through Bolts. Use through bolts to maximum extent feasible (for ease of replacement).

Section 5. SPECIAL CONSIDERATIONS

1. SERVICE LIFE. The provisions of DM-2.1 relating to service life (25 years) shall apply, except:

a. Estimating Service Life. Structures detailed in accordance with this manual shall be assumed to meet the above requirement.

b. Accessibility. Since waterfront structures commonly are required to serve longer than 25 years, detail so that all areas or components with an anticipated service life of less than 50 years can be inspected and repaired.

c. Breakwaters. Breakwaters and other constructions whose design is controlled by wave forces shall be proportioned to resist a design wave of a 50 year interval of recurrence.

2. CORROSION OF STEEL PILING.

a. Principal Factors Affecting Rate of Corrosion Loss.

(1) Geographical location.

(2) Zones relative to tidal planes.

(3) Exposure to salt spray.

(4) Sand, earth, or other cover.

(5) Protective coating.

(6) Abrasion conditions (surf zone vs. deep water).

(7) Stray electric currents.

(8) Type of soil.

b. Rate of Corrosion Loss. See Table 1 for some experimentally derived corrosion loss rates. See Figures 4 and 5 for some actual measured profiles. To estimate Service Life, refer to Table 2 for approximate periods of protection which can be expected to be afforded by various coating systems and estimate rate of loss of metal thereafter from Table 1, Figures 4 and 5, or from other available experience.

c. Tropical Climates. Steel piling (carbon-manganese steel) should be faced with concrete in and above the tide range and to a minimum of 2 feet below mean low water. Steel sheet piling should be capped with concrete and not with a steel channel or timber.

d. Use of Weathering Steels.

(1) Do not use alloys conforming to ASTM A242, A588, or A690 in a marine environment, unless boldly exposed to wind, rain, and sun. Otherwise protect as if it were plain, carbon steel.

(2) If an alloy conforming to ASTM A690 is used, hardware should conform to ASTM-A588.

(3) Copper-bearing steels (ASTM A709) Grades 50W and 100W, including compositions known as "Mariner Steel" (ASTM A690), require coating in the splash zone and other areas not boldly exposed to sun, wind, and rain. There is no data on the rate of corrosion loss from surfaces in contact with the soil, so that considerat6on should be given to coating such surfaces in the same manner and degree for carbon manganese steel (ASTM A36).

3. CATHODIC PROTECTION. The desirability of providing cathodic protection for waterfront structures requires careful consideration, including the following factors:

a. Efficacy. In general, the rate of corrosion loss below MLW is two-thirds to one-half the rate just below, at, and above MLW. Since cathodic protection is effective only below MLLW, it follows that cathodic protection should be accompanied by use of concrete facing or encasement to and below MLLW. Alternately, future repair by partial jacketing must be considered in the economic analysis.

b. Maintenance Cost. Consider cost of electricity, replacement of anodes, and general repair of damage to wires and hangers in the economic analysis.

TABLE 1

CORROSION RATES OF H-PILES BASED ON FLANGE THICKNESS MEASUREMENTS[a]

Coating Description	Average Corrosion Rate Within Zone (mils per year)			
	Imbedded Zone 0 to 15 ft.[b]	Erosion Zone 15 to 21 ft.[b]	Immersed Zone 21 to 29 ft.[b]	Atmospheric Zone 29 to 35 ft.[b]
Coal Tar Epoxy/Zinc Rich Inorganic	<0.01	<0.01	<0.01	<0.01
Vinyl/Flame Sprayed Aluminum	0.01	0.17	0.07	0
Epoxy Polyamide/Zinc Rich Inorganic	0.02	0.22	0.10	0
Aluminum Pigmented Coal Tar Epoxy	0.07	0.06	0.08	0.03
Polyester Glass Flake	<0.10	<0.10	<0.10	<0.10
Polyvinylidene Chloride/ Flame Sprayed Zinc	0	0.14	0.12	0.29
Galvanized	0	0.67	0.32	0.06
Phenolic Mastic	0.11	0.11	0.15	0.21
Flame Sprayed Aluminum	0.19	0.39	0.19	0.03
Aluminum Pigmented Coal Tar Epoxy	0.18	0.08	0.21	0.04
Coal Tar Epoxy/Zinc Rich Organic	0.17	0.15	0.71	0.24
Vinyl/Zinc Rich Inorganic	0.19	0.22	0.18	0.31
Vinyl Mastic/Zinc Rich Inorganic	0.02	1.4	0.61	0
Coal Tar Epoxy	0.17	0.21	0.27	2.1
Coal Tar Epoxy on Mariner Steel	0.18	0.44	0.45	1.6
Coal Tar Epoxy plus Armor	0.13	0.07	0.07	2.7
Coal Tar Epoxy	0.27	0.72	0.46	2.9
Vinyl-Red Lead/Flame Sprayed Zinc	0.08	3.2	1.8	2.3
Polyvinylidene Chloride	0.81	4.9	3.6	3.5
Bare Carbon Steel	0.9	8.9	6.7	10.5
Bare Carbon Steel	1.8	9.7	7.9	12.2
Bare Carbon Steel	2.8	10.5	9.0	13.9

(a) Location: Atlantic Ocean, Dam Neck, Virginia.
Piles were 35 feet in length with 19-feet buried below mudline (sand). MLW was 4 feet above mudline and MHW 10 feet above mudline.

(b) Distance from bottom of pile.

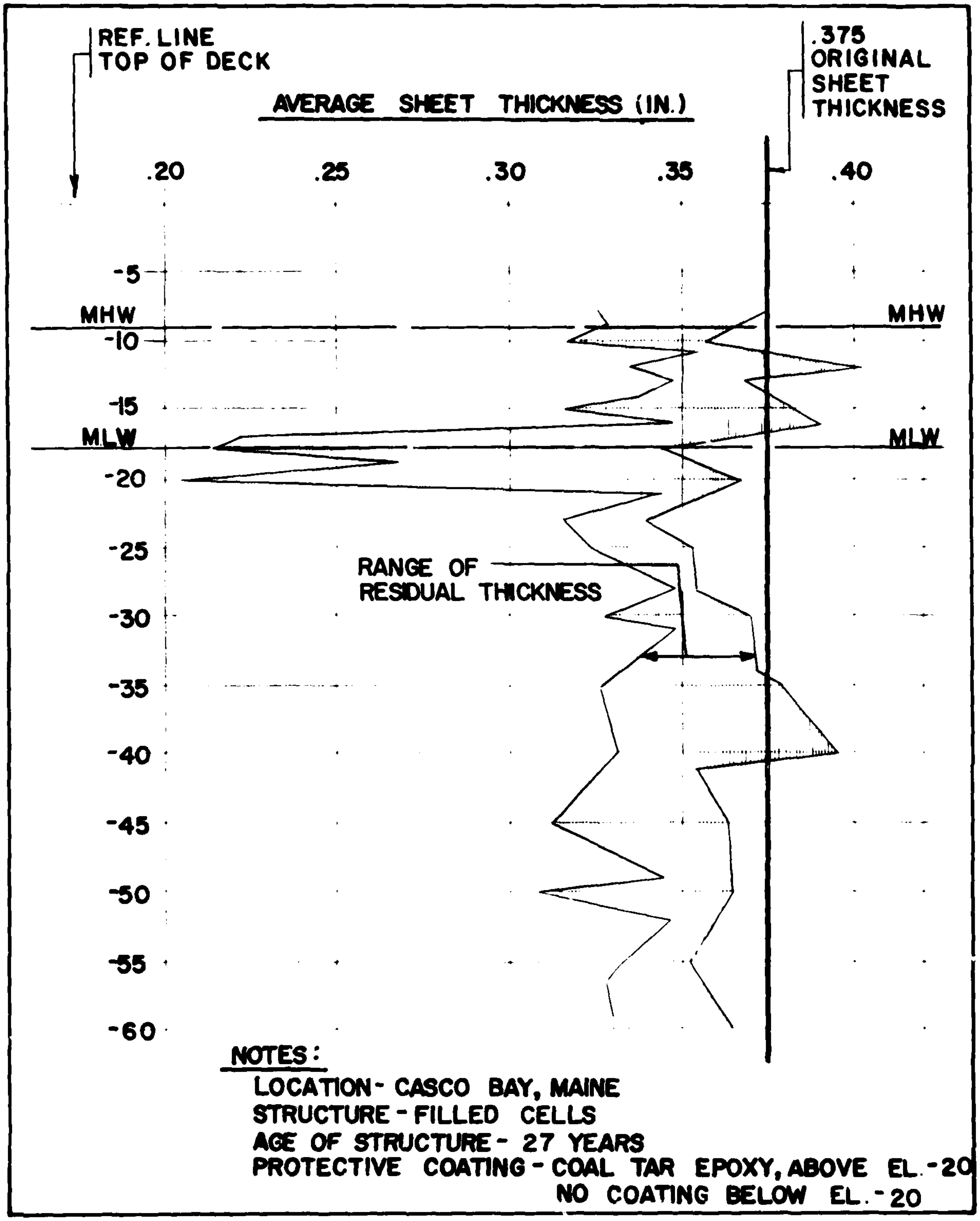


FIGURE 4

Measured Profile Envelope-Steel Sheet Piling

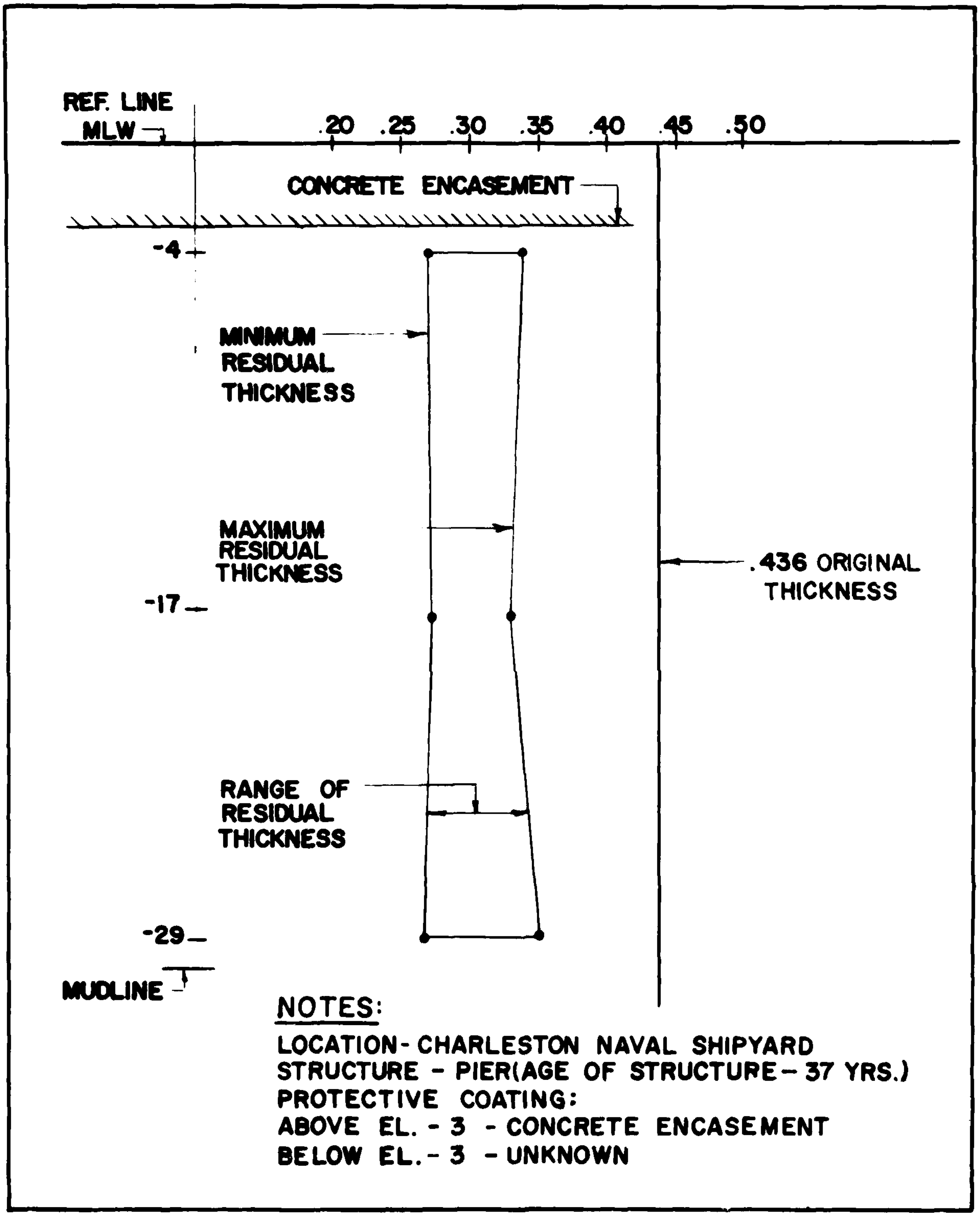


FIGURE 5
Measured Profile Envelope-Steel H-Piling

TABLE 2

PERIODS OF PROTECTION FOR STEEL
TO BE EXPECTED FROM VARIOUS COATING SYSTEMS OF COMMON USE[a]

Coating Description[b]	Period of Protection[c]
Coal tar epoxy (15 to 20 mils thickness)	10 - 20 years
Galvanizing (7 to 9 mils thickness)	10 - 15 years
Metallized Aluminum	15 - 20 years
Concrete Encasement	25 years

(a) Marine exposure

(b) Coatings applied properly

(c) Periods of good to excellent protection, i.e., negligible loss of metal.

c. Reliability of Maintenance Effort. A fully or partially inoperative cathodic protection system due to a lack of maintenance and repair is an all too-frequent observation.

d. Coating Systems in Lieu of Cathodic Protection. Highly effective coating systems exist. The use of coatings, in lieu of cathodic protection, warrants consideration. Bear in mind that, in general, for structural considerations it is overall loss of sectional area or section modulus which is important and that pitting is not a concern. Accordingly, objections to certain types of coatings (i.e. holidays and small damages) which would be valid regarding their use for pipelines are not significant as regards piling.

4. EFFECTS OF BULBOUS BOWS AND PROJECTING PROPELLER GUARDS. Many modern vessels have a bulbous bow. Theoretically, with careful maneuvering, these protrubences should not strike the fender system or project under the pier. However, their effect should be considered in the design of the fender system, in the design and location of the outboard line(s) of piles, and in the design of a berthing bulkhead. Where feasible, the best solution is to overhang the fascia or to use camels. Figure 6 and Table 3 are provided to illustrate the amount of underrun of the bulbous bow of various commercial vessels approaching the berthing face at various angles. Although the projections of destroyers, cruisers, and frigates are greater in the lateral direction than those shown in Figure 6, underrun is not of concern for these naval vessels because the projections are located below and behind the waterline bow profiles. The exceptions are frigates of the FF 1040 and FF 1052 classes, which have projections extending laterally beyond the hull. For projections of propeller guards and stern planes for submarines, consult NAVFAC DM-26.6, "Mooring Design Physical and Empirical Data." In general, submarines are berthed using camels so that the underrun problem is not of concern.

5. FORCES DUE TO CURRENT AND PROPELLER SWASH. Normally, current forces are neglected in the design of harbor structures. However, the rational design of exposed piling as a column, as described in Section 2, requires that lateral forces due to current be considered. The following formula may be used when estimating current forces:

Equation: $$P_{(\text{lbs./S.F.})} = 1.5\ V^2 \qquad (5\text{-}1)$$
with V in ft./sec.

The coefficient 1.5 provides for roughness due to organic growth and for the resulting shape of the pile. Increase the effective diameter of the pile for effects of organic growth. When designing the bearing piles within 20 feet of a berthing face, assume a current velocity generated by the propellers of tugs or departing vessels to be 8 knots.

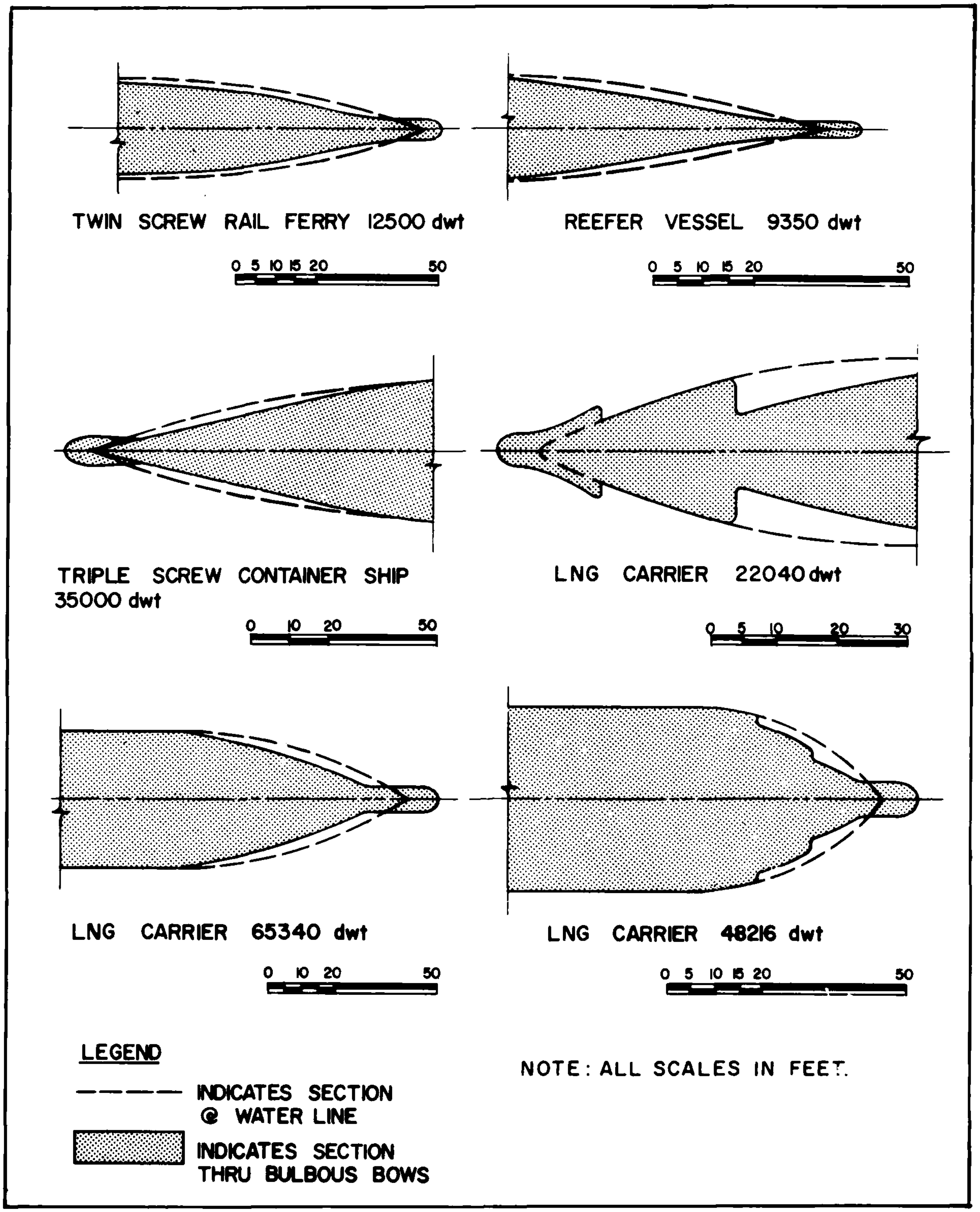


FIGURE 6
Hull Configurations, Ships with Bulbous Bows

TABLE 3

EXTENSIONS UNDER PIER (BULBOUS BOWS)

Vessel	Angle of Approach (Degrees)			
	5	10	20	30
9350 dwt reefer vessel	-	0.62m[a] (2.03 ft.)	2.5 m. (8.20 ft.)	3.72 m. (12.20 ft.)
12,500 dwt twin screw rail ferry	-	-	2.0 m. (6.56 ft.)	3.5 m. (11.5 ft.)
22,040 dwt LNG carrier	-	-	2.4 m. (7.87 ft.)	3.75 m. (12.3 ft.)
35,000 dwt triple screw container ship	-	-	2.6 m. (8.5 ft.)	3.0 m. (9.85 ft.)
48,216 dwt LNG carrier	-	-	-	1.75 m. (5.75 ft.)
65,340 dwt LNG carrier	-	-	2.75 m. (9.0 ft.)	5.5 m. (18.0 ft.)

(a) m. = meters

6. EXPANSION, CONTRACTION, AND CONTROL JOINTS.

a. Open-Pile Platforms. (See DM-25.1.)

b. Bulkheads. In normal practice, no expansion or contraction joints are provided in the sheeting (whatever form it may take). However, expansion joints are provided in the concrete cap and encasement at approximately 30 ft. centers. If a timber or steel channel cap is used, no movement joints are provided, but adjacent sections of the cap are not connected. No movement joints are provided in the anchor wall.

c. Quaywalls. Movement joints shall be provided at approximately 300 ft. spacing. Such joints need not be carried through the sheet piling nor do they need to extend more than 5 ft. below low, low water.

7. MISCELLANEOUS REQUIREMENTS.

a. General.

(1) Service Life. The actual service life of waterfront structures often far exceeds the specified design service life of 25 years. During this period, types and size of ships and usage changes. One of the most important aspects of the design of berthing facilities is flexibility to accommodate multi-purpose usage, such as: (1) a mix of sizes and types of ship (submarines and carriers are exceptions and often require dedicated berths), and (2) nesting. A second, important consideration is to detail the design to provide ease of maintenance and repair of both the structure and the utility services.

(2) Protective Lighting. Consider the need for protective lighting in the following areas. Lighting levels are to be as stated in OPNAVINST 5510.45B.

(a) Land approaches to piers and docks

(b) Water approaches

(c) Decks of open piers

(d) Underside of platform decks of piers and wharves, including trestles.

(3) Emergency Power. In general, any security area provided with protective lighting should have an emergency power source located within that security area. (See OPNAVINST 5510.45B).

(4) NAVAIDS. Provide NAVAIDS at ends of pier, wharf, or quay. Cost of prominent, well-lighted markers is negligible compared to the cost and hazard of a collision. Consult DM-26.1 "Harbors" for specific requirements.

(5) Fender System. Even though not intended for berthing, provide a fender system on all piers, wharves, bulkheads or quaywalls in consonance with the size of vessel which could be berthed (as limited by the depth of water and the length of the berthing face).

b. Structural.

(1) For bulkheads, where feasible, schedule dredging operations after the bulkhead sheeting is in place and tied back. For pile-supported platform structures, schedule dredging operations to precede the pile driving in order to minimize disturbance to piles already in place.

(2) Unless positive restriction can be assured, assume that at sometime during its service life a mobile crane will be used on the deck of a waterfront structure. Even if not part of the project criteria, design for usage by a mobile crane of capacity specified in DM-25.1 as applicable to the type of pier being considered. Use a load factor of 1.15 if rare and incidental use is intended, or higher load factors if more frequent use is judged to be probable.

(3) Recess hold-down bolts in bases of mooring fittings to avoid interference with lines. Fill pockets with lead.

(4) To the extent feasible, use through bolts in sleeved holes for fastening mooring fittings and fender system components to structures.

(5) Allowable tension in concrete in a seawater environment is the same as in an upland environment (ACI 318).

c. Utilities.

(1) If feasible, do not place service lines below deck level unless in trenches or in ballast. If use of trenches or ballast is not practical or is grossly uneconomical, lines must be protected from corrosion by high-performance coatings. Do not place service lines where they will be directly exposed to salt water or to salt spray unless protected by coatings. This includes outlets and connections. Flush deck service outlets of the type shown in Figure 7 have given good service. Where outlets and connections must protrude above the deck level, cover them in a manner which will provide personnel safety and preclude securing mooring lines to the piping. (See Figure 8 for suggested details.) Do not place service lines where they will be subject to impact of drift. Where exposed underdeck lines must be used, locate them high enough or behind protecting piles to avoid impact damage from floating ice or debris. Work platforms and access shall be provided at underdeck valves and other operating fittings.

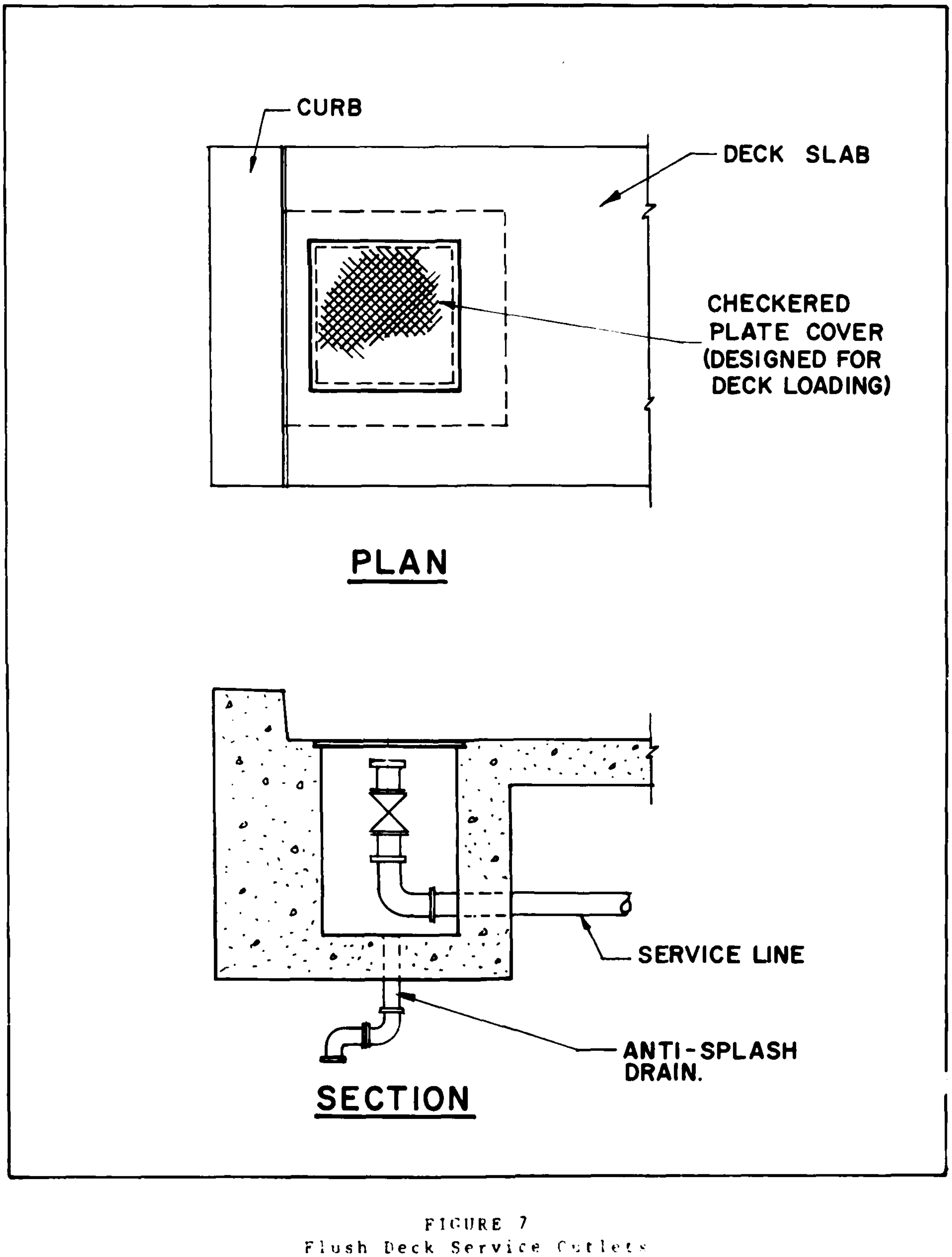


FIGURE 7
Flush Deck Service Outlets
25.6-31

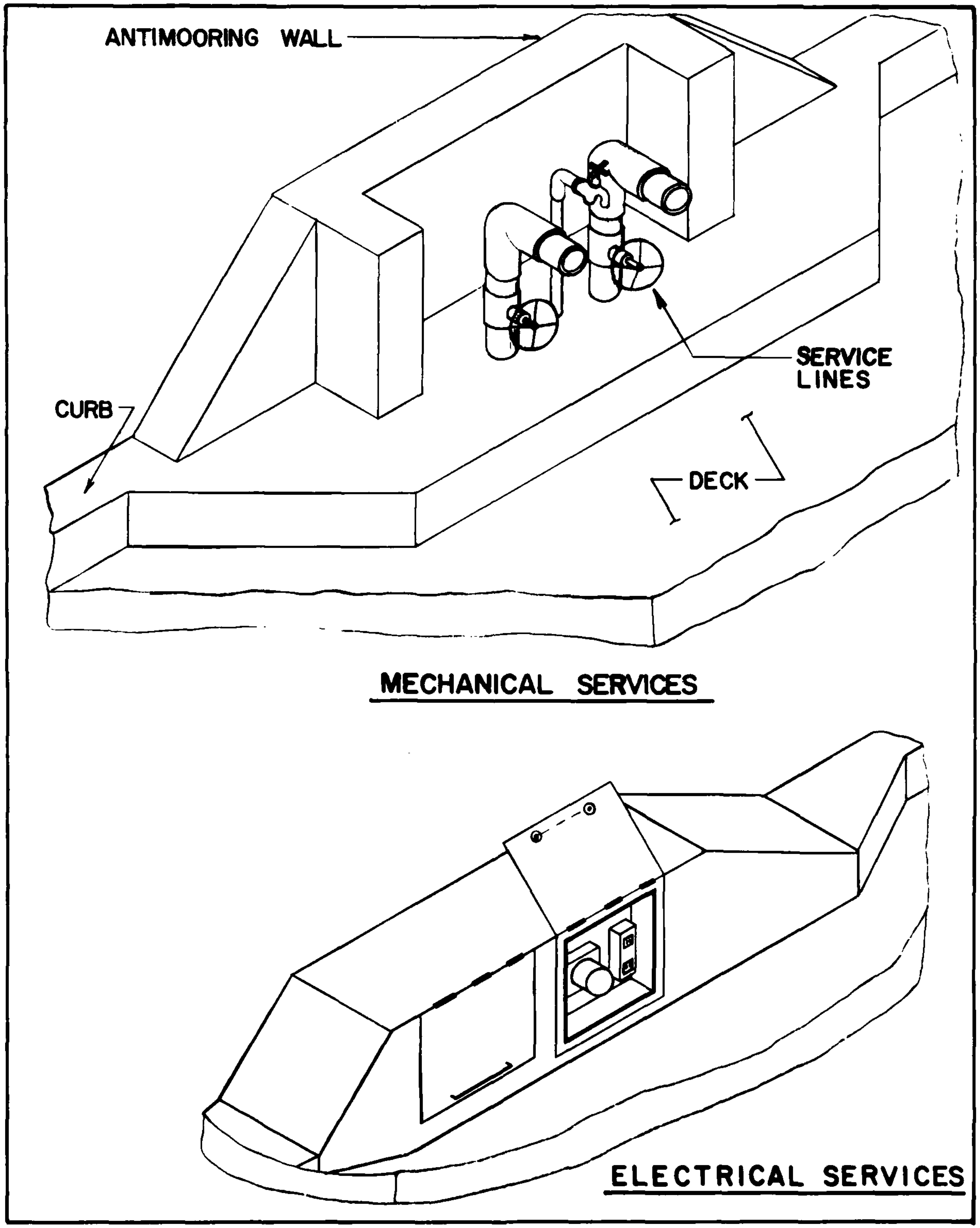


FIGURE 8
Protection of Utilities Above Deck Level
25.6-32

(2) Make layouts of all proposed and potential future services to check for fit, interferences, and accessibility. Experience indicates that provision for future installation of additional lines is worthwhile if the cost of doing so is not excessive. Additional size of trench, additional size in utility stations, room for expansion of substations, and most important, sleeves and clearance should be considered. The assumed configuration of hoses and like connections from utility stations on the structure to stations on the ships shall be included. Check to see if the weight of the hose or cable will require a jib crane, or boom, or other special device for handling.

(3) Security measures shall be provided to prevent unauthorized access to key items of equipment such as switchgear, fire pumps, pump wells, or caisson gates.

(4) For underdeck lines, place drain valves in accessible locations to avoid climbing over the side of the platform.

(5) Access hatches in decks are to have flush-mounted covers and are to be detailed to eliminate tripping hazards.

d. Dredging Under Platforms. A set of soundings taken under an open pile platform shortly after dredging, or after some years of accretion of silt, typically looks like Profile A in Figure 9. Particularly for closely spaced bents, the restraint offered by the piles results in a slope steeper than the normal angle-of-repose. Eventually, this material will slough, or wash down into the slip. If the material is soft or loose, or for granular soils, it flows around the piles and no harm results other than additional maintenance dredging. The lateral loads on the piles due to movement of the soil are small. For cohesive soils, however, blocks of material may tend to move, or a mass failure (curve B of Figure 9) may develop, either of which can exert substantial lateral forces on the piles, which the piles are ill-suited to resist. Accordingly, at least for cohesive soils, washing down of the soil under the pier should be specified every few cycles of maintenance dredging. Initial dredging should include a controlled slope under the platform (not just dredge up to the fascia and let the soil fall in), and the initial dredged slope should be flattened as shown in Figure 9 to meet the daylight line of probable future dredging.

8. DETAILING FENDERING SYSTEM TO RESIST EFFECTS OF ROLLING OF THE VESSEL. A fender system is designed to resist the impact of an approaching vessel. Commonly, the impacting blow is assumed to occur flat on to the fendering, i.e. assuming no list on the vessel. This is not always correct. Further, when berthed alongside, the vessel rolls. The consequence of both of these events is that certain details, such as those shown in Figure 10, work poorly. A fender system should be detailed on the assumption of 5 degree roll or list of the berthed, or berthing, vessel.

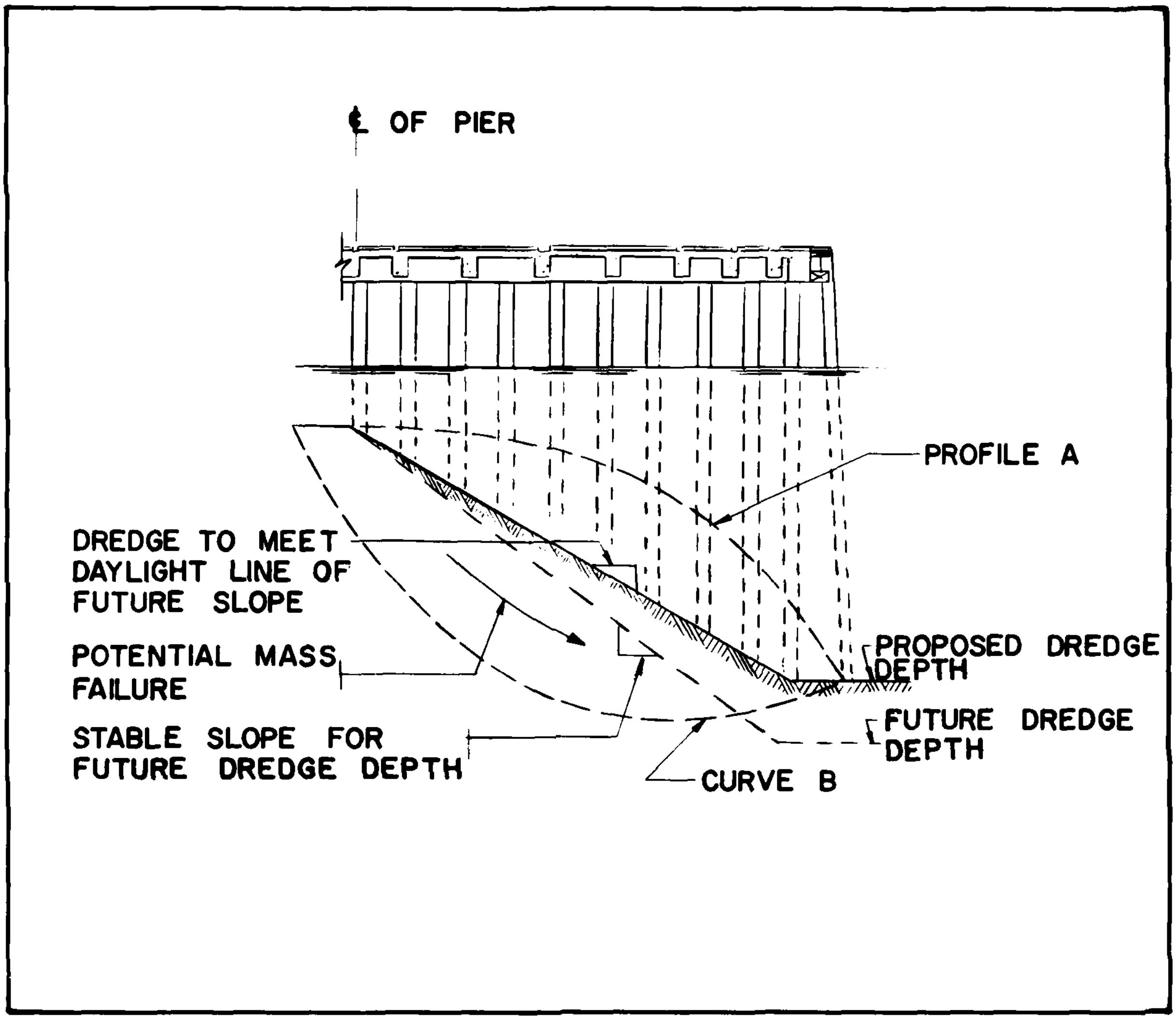


FIGURE 9
Dredging Under a Platform

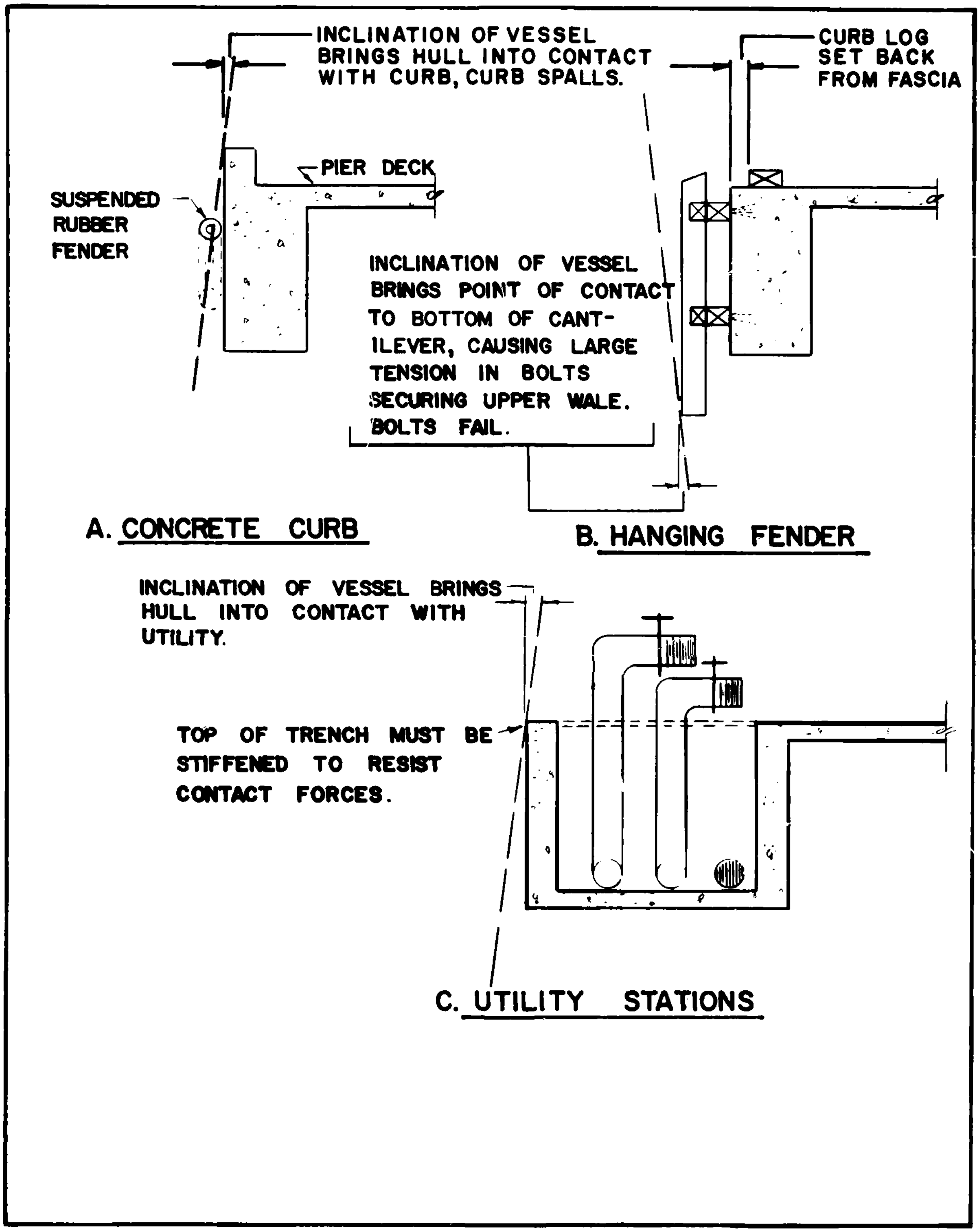


FIGURE 10
Some Details Which Are Affected
By List or Roll of Berthing (Berthed) Vessels

9. STRENGTH OF WOOD WHICH HAS BEEN GIVEN A PRESERVATIVE TREATMENT. For treated douglas fir or southern pine (piles or sawn timbers), reduce strain, strength, and energy absorption properties as indicated in Table 4.

10. FIRE PROTECTION REQUIREMENTS.

a. General. The provisions of Construction and Protection of Piers and Wharves, NFPA No. 87 and of DM-8 shall apply. The following supplementary rules have been assembled for ease of reference:

b. Supplementary Rules.

(1) Do not rely on low pressure water connections as an adequate fire protection system.

(2) The fire protection system shall be capable of use for fighting fires on and in vessels berthed at the pier, wharf, or bulkhead, as well as a fire on and under the structure.

(3) Fire protection system shall be tailored to the type and condition of vessels being berthed. For example, a salt water system would be inappropriate at most submarine berths.

(4) Do not use flush deck-type valved outlets in locations subject to accumulations of ice and snow.

(5) Where wood deck or wood piles are used, provide firewalls across the width of the pier at a maximum of 150 foot intervals. Firewalls are to extend from the underside of the deck to low, low water.

(6) Whatever the materials of construction (including noncombustible materials), provide 6 inch diameter holes (minimum) with removable covers and, where feasible, provide asbestos-cement liner in the deck to permit insertion of spray nozzels to fight below deck fires such as those due to floating, burning oil. Locate these holes on a 25 foot maximum grid.

(7) Detail the fender system to permit access at intervals not exceeding 150 feet in order to fight floating fires. Access manholes in the deck are not desirable.

(8) If a shed is provided, do not support the shed columns on timber grillages or piles unless the use of timber is restricted to areas below the level of permanent saturation (generally about mid-tide level). Build-up about this level with a concrete pedestal.

(9) Provide a standpipe line in a pier shed so that fire-fighting equipment need not go into the pier shed to fight a fire.

(10) Consider shotcrete or concrete jacketing for wood piles (to MLW).

TABLE 4

PROPERTIES OF TREATED WOODS

Type of Treatment	Average Properties							
	Modulus of Rupture		Modulus Elasticity in Flexure		Average Absorb. Energy in Flexure		Compressive Strength F_c	
	(psi)	%	(10^6 psi)	%	(in-lb/cu.in.)	%	(psi)	%
Fir								
Untreated	8,394	100	1.922	100	6.388	100	3,346	100
Creosote	6,862	82	1.584	82	4.202	66	-	-
ACA dual	6,111	73	1.537	80	3.059	48	2,714	81
CCA dual	3,844	46	1.171	61	3.364	53	2,333	70
ACA	5,620	67	1.416	74	2.078	33	2,462	74
Pine								
Untreated	8,007	100	1.942	100	5.240	100	-	-
Creosote	5,950	74	-	-	-	-	-	-
ACA dual	4,725	59	1.568	81	2.829	54	-	-
CCA dual	4,167	52	1.441	74	2.413	46	-	-
ACA	5,534	69	1.538	79	-	-	-	-
CCA	5,410	68	-	-	-	-	-	-

Notes:
1) Where no value is provided it is because of the large spread in measured values for a small number of samples.
2) % = the percent of the value for untreated wood.
3) Source: Civil Engineering Laboratory, Technical Note No. N-1535 "Mechanical Properties of Preservative Treated Marina Piles - Results of Limited Full - Scale Testing."

Section 6. STRENGTH EVALUATION OF EXISTING WATERFRONT STRUCTURES

1. EVALUATION OF STRENGTH OF EXISTING MATERIALS.

a. General. The provisions of DM-2.1, relating to the use of used and unidentified materials, shall apply.

b. Number of Tests Required to Establish Strength of Ungraded Materials. Where documentary data as to quality of the existing material is lacking, the strength shall be established by tests of the material. The strength of material to be assumed for strength evaluation of the structure shall be the value which sampling and test indicates to have a 95 percent probability of being exceeded. Not less than 4 samples shall be tested.

(1) Calculate strength of material for a given set of tests as:

Equation:
$$S = x - \frac{\sigma t}{2} \qquad (6\text{-}1)$$

WHERE: x = average (arithmetic mean) of tests

σ = standard deviation of the test values. A normal distribution curve may be assumed.

t = a factor (see Figure 11)

c. Tests and Test Specimens.

(1) For steel members take test specimens from locations, and as described in, ASTM A6, "Standard Specification for General Requirements for Rolled Steel Plates, Shapes, Sheet Piling and Bars for Structural Use."

(2) For concrete members, use of drilled cores and sawed beams shall be as described in ASTM C42, "Obtaining and Testing Drilled Cores and Sawed Beams of Concrete."

(3) For wood members, stress grade visually as described in ASTM D245, "Standard Methods for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber."

2. COMPUTATION OF STRENGTH OF THE STRUCTURE. Analysis shall be based on measured in-place dimensions and as-built conditions. A badly deteriorated or obviously overloaded structure often continues to support the applied loads with no discernible indications of distress. It is important to consider the factors contributing to this

phenomenon when evaluating the strength of an existing structure. The more important of these factors are:

a. Simplifying Design Assumptions. Structural design commonly employs simplifying assumptions intended to make the design effort more manageable. These assumptions, necessarily, are conservative. Often, they leave substantial excess strength. Some examples are:

(1) Distribution of Concentrated Loads to a Slab. Conventional procedures, such as those described in the references of DM-2.3 and 2.4 (AASHTO), grossly underestimate this distribution. Use grid analysis (and computer) to obtain more realistic answers.

(2) Variable Section. Figure 12 shows the cross section of a pier supported on steel "H" piles. The upper portion of the piles was jacketed with concrete to protect against corrosion. The stiffening effect of the jackets reduced the L/r by 10 percent and increased the structural capacity of the piles by 25 percent.

b. Locations of Weakened Sections. Members are proportioned for maximum stress conditions. The section required at points of maximum stress frequently is carried for the full length of the member to minimize the costs of fabrication or of form work, or for aesthetic reasons. If the deterioration of a member is localized and does not occur at a point of maximum stress, the strength of the section may not be impaired. For example:

(1) The piles shown in Figure 12 had suffered substantial corrosion loss in the level just below the bottom of the jackets. Loss of section up to 40 percent was measured. However, the magnitude of loss was limited to a distance of a few feet. Below this area, corrosion attack was less. Analysis of column capacity based on actual, measured sections indicated capacities were 20 percent or more, greater than if the maximum loss had occurred over the full length of piles, or at the middle of the buckling length.

(2) Figure 13 shows a stringer of a trestle. Leakage through the expansion joint over the abutment has corroded the flange until there is little section left. However, there is negligible moment at this section. Accordingly, the loss of flange material is not critical.

c. Changed Design Standards. If the design is based on an elastic analysis, reanalysis on the basis of ultimate strength, of plastic redistribution of moments, or of relief of moment based on the concept of the yield line frequently will indicate a greater capacity.

d. Design Live Loads. Design live loads are seldom realized in practice. The actual and the design loading conditions should be compared.

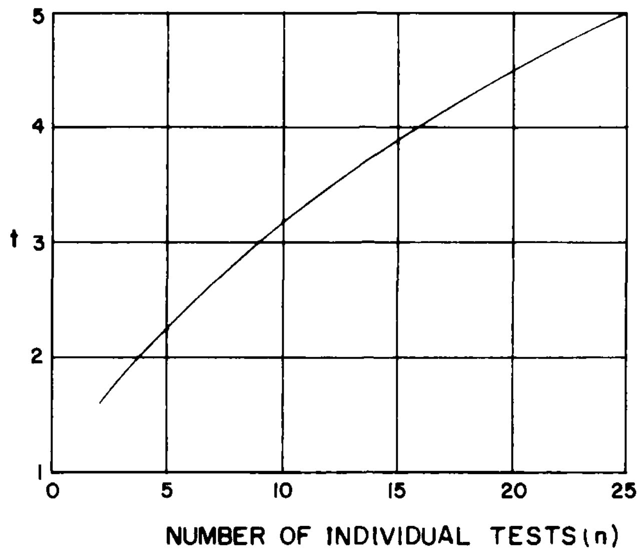


FIGURE 11
Values of "t" for
a Given Number of Individual Tests

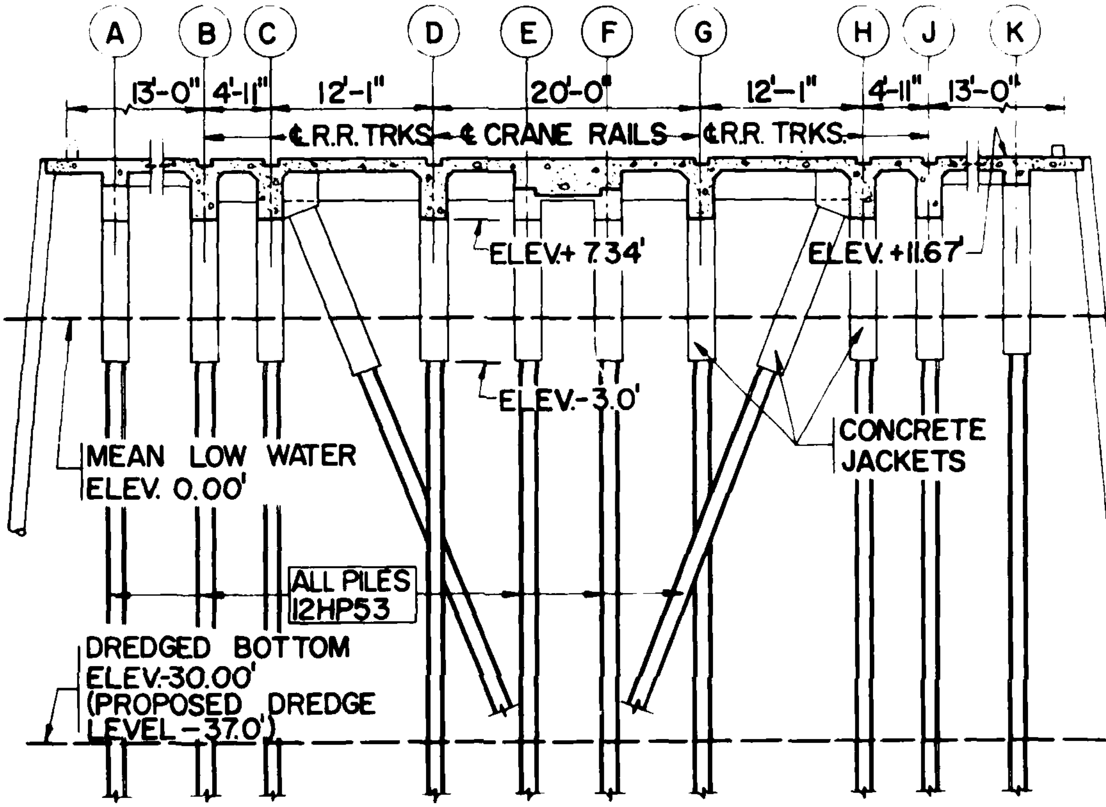


FIGURE 12
Pier on Steel H-Piles

FIGURF 13
Corrosion of a Stringer

e. Excess Section. The design may incorporate excess strength by way of sacrificial metal, by sizing the members to the next heavier section, or to a lighter but stronger section, or by satisfying requirements for minimum thickness of metal or limiting deflection. Piling, in particular, often is sized to resist driving stresses (or for load transfer to the soil). Static stresses are much less.

f. Increased Strength of Concrete With Age. An increase of 30 percent, after 2 years, compared to the 28 day strength, would be a reasonable expectation. The resulting increase in moment capacity would be about 6 percent.

g. Change of Structural Action. The structural action may differ from that assumed for purposes of design. Ordinary beams and slabs are a common case in point. These are proportioned on the basis of flexural behavior. However, except for large ratios of span to depth, pure flexural action is not achieved, and the member resists the load, at least partly, by arching or catenary action. (See Figures 14 and 15.) Composite action may develop, which was absent in the original construction. (See Figure 16.) Yield points may develop, changing the moment diagram and reactions, thus increasing some and decreasing others, with the changes often being non-critical. (See Figure 17.)

h. Change in Loading. In some cases, the design load represents a temporary or construction condition, and the service loads are of lesser magnitude. For example, consider a retaining wall. If the wall is well drained, maximum lateral pressure will occur during and shortly after backfilling with the active pressure decreasing with time. Another example is that of a hydraulic fill. The lateral pressure decreases as the fill drains. Borings will help in evaluating actual, in-place soil properties at the time that evaluation is made.

3. EVALUATION OF STRENGTH BY USE OF LOAD TESTS.

a. Method. The provisions of Chapter 20 of ACI-318 shall apply, supplemented as described in the following paragraphs.

b. Test Load. Use 1.4 D + 1.7L, where live load is the reduced load (for tributary area). This increased loading intensity will require careful observation and control to preclude precipitating collapse. For this purpose, load in 6 increments, rather than 4 and, where feasible, use water loading with provision for emergency drainage.

c. Lateral Loads. Lateral loads which are simultaneous with vertical loads shall be simulated in the test.

d. Loaded Area. The loaded area shall be large enough that reserve of strength due to continuity and three dimensional action of the structure is properly reflected in the test.

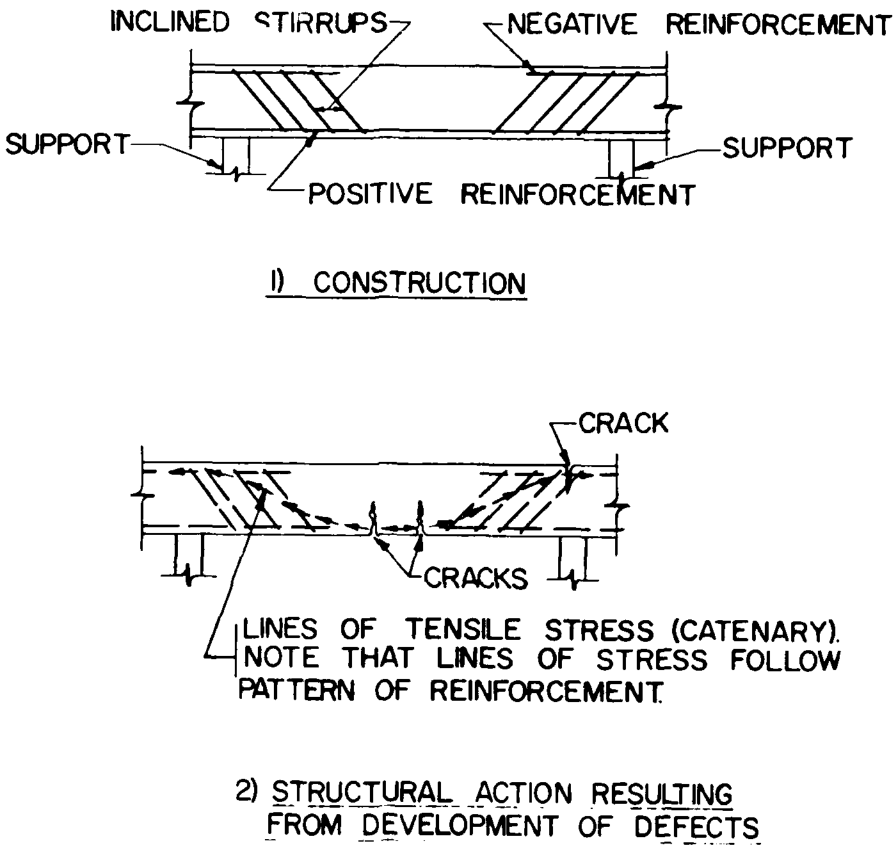


FIGURE 14
Catenary Action Due to Cracking of Reinforced Concrete Member

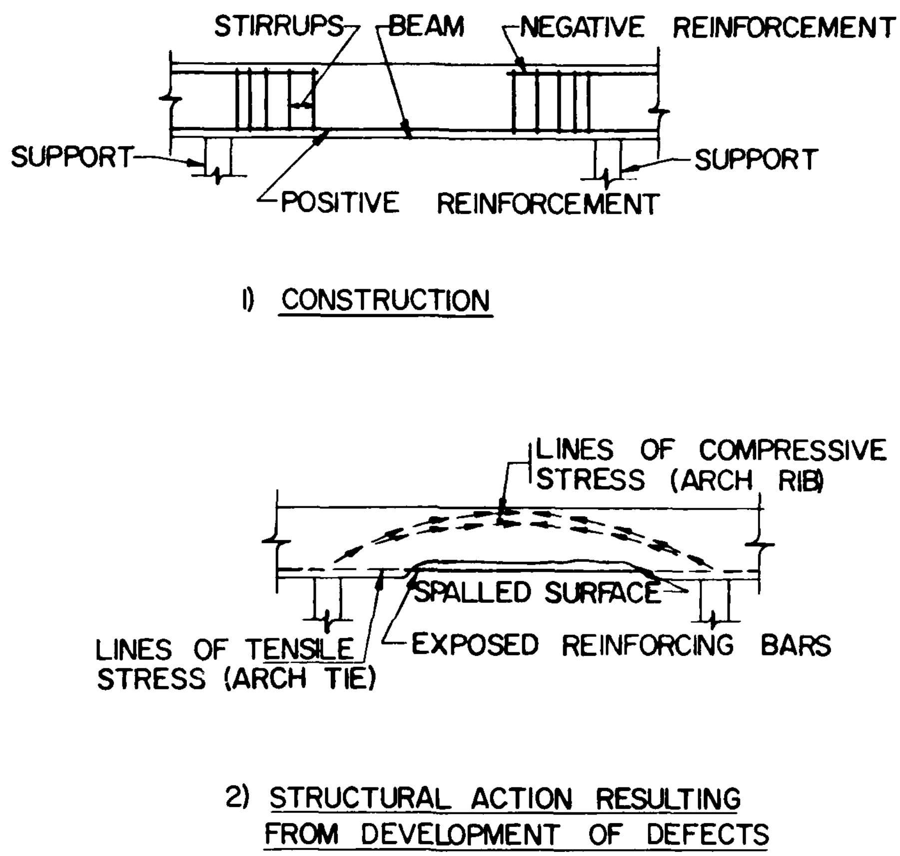
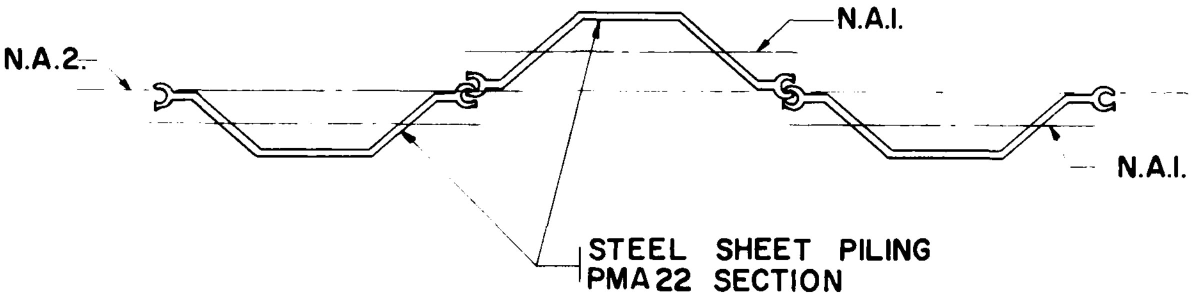


FIGURE 15
Arch Action Due to Spalling of Reinforced Concrete Member



NOTES:

1) DESIGN STRENGTH PREDICATED ON EACH SHEET PILE ACTING INDIVIDUALLY. SECTION MODULUS = 5.4 $in.^3$ PER FOOT OF WALL (PMA 22 SECTION)
2) IF INTERLOCKS RUST TIGHT AND CAN DEVELOP SHEAR, NEUTRAL AXIS SHIFTS TO N.A.2 AND SECTION MODULUS IS INCREASED TO 7.1 $in.^3$ PER FOOT OF WALL, AN INCREASE OF 32%.
3 STEEL SHEET PILING HANDBOOK RECOMMENDS USE OF SECTION MODULUS OF 6.8 $in.^3$ PER FOOT OF WALL (PMA 22 SECTION).

FIGURE 16
Increase of Strength of Sheet-Pile Wall Due to Aging

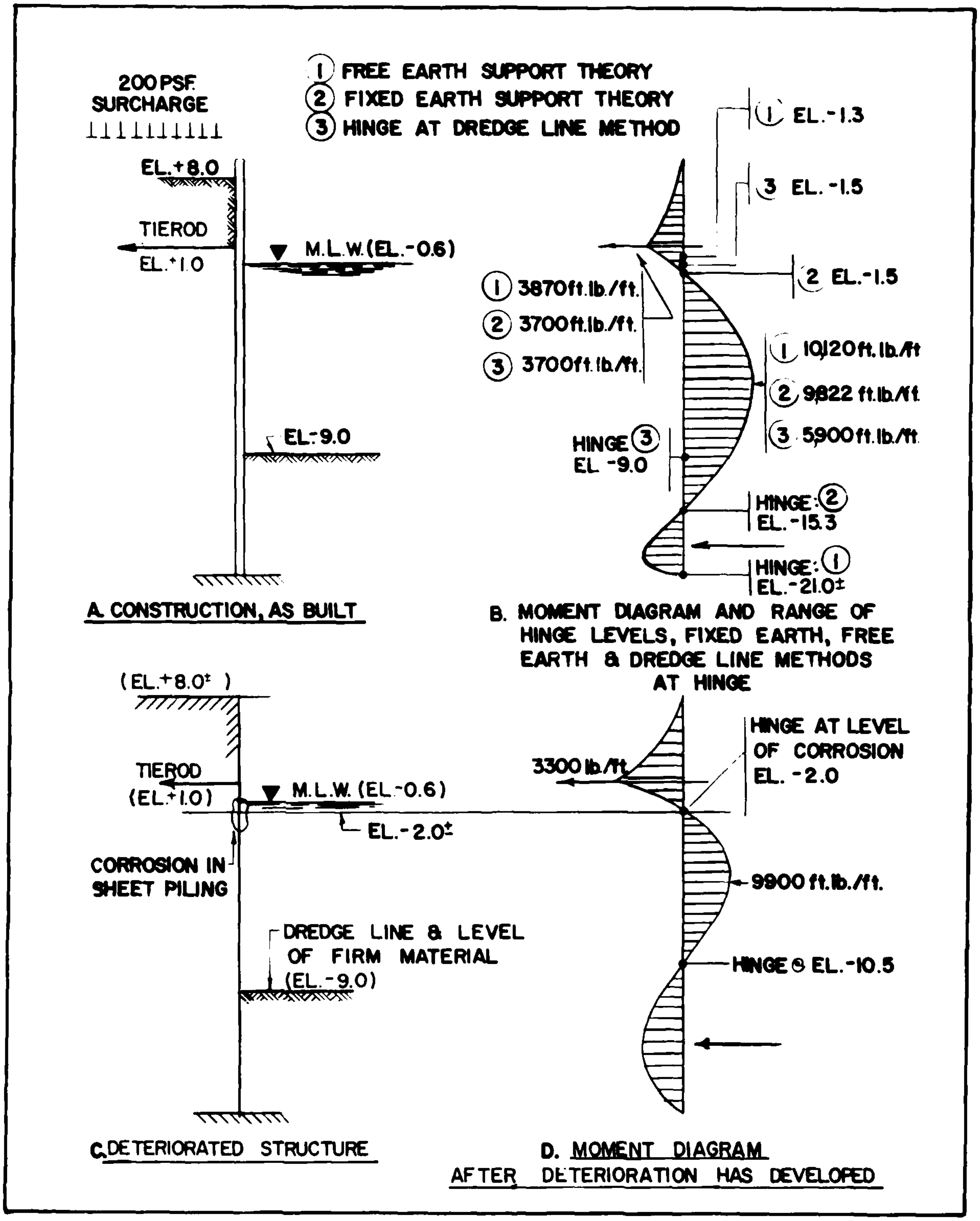


FIGURE 17
Change in Flexural Action in Bulkhead
After Deterioration Has Occurred

4. SPECIAL PROVISIONS REGARDING CAPACITY OF EXISTING PILES.

a. Structural Capacity. Check for effects of deterioration. A reconnaisance survey should be made to identify areas of "worst conditions". Measurement of overall residual strength of 1 percent to 2 percent (but not less than 4) of the piles will be considered as an adequate statistical sample on which to base judgement of capacity. These shall be the "worst" piles of the group, as identified in the reconnaisance survey. Consideration shall be given to probable, future progression of loss of strength. It often will be found that the mud line under a platform has accumulated well above the normal, stable slope line drawn from the existing dredge level alongside the platform. This material should be discounted in estimating the unbraced pile length (L). Should future dredging to greater depth be contemplated, consider the increased pile length which would result.

b. Capacity of the Soil to Support the Pile. Unless special circumstances prevail, for example, loss of support due to dredging, assume no change from capacity as installed. Where installed capacity is not known, consider the use of load tests to establish capacity.

c. Sheet Piling. The capacity of sheet piling to support vertical loads shall be taken as one-half the value indicated by conventional formulae relating capacity to driving resistance.

d. Intrepretation of Load Tests. (See DM-7.)

5. STRENGTHENING AN EXISTING STRUCTURE.

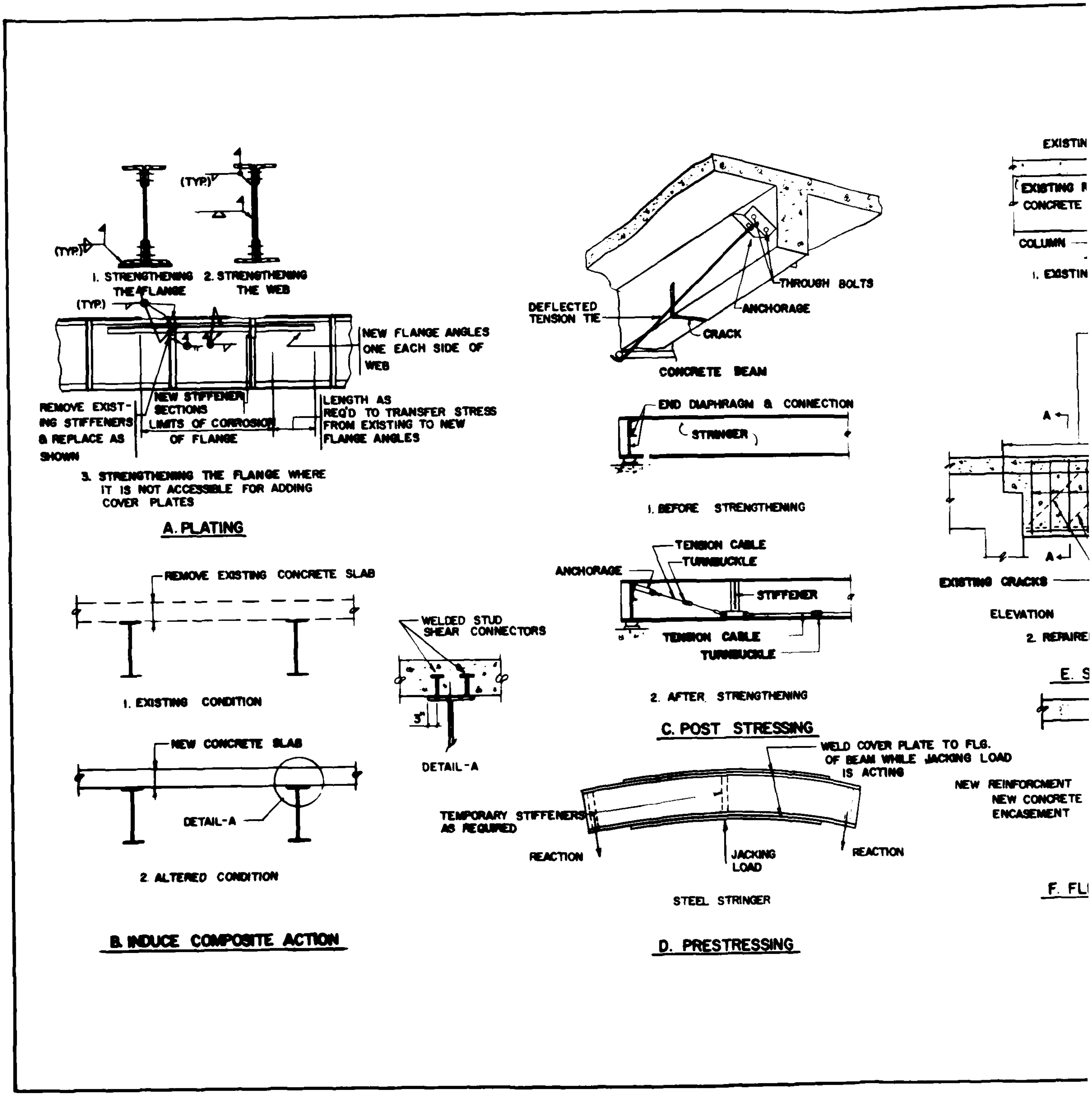
a. Methods. (See Figure 18.)

(1) Plating. Where the top of flange is not accessible for adding cover plates, reinforcement can be added to the web plate. The beam preferably should be relieved of load before the reinforcement is added. When cover plates are added, the flange to web connection and the web plate stresses at the toe of the flange should be investigated.

(2) Composite Action. Beam section properties can be materially increased by causing the concrete slab to act compositely with the beam. The slab serves as a top cover plate.

(3) Prestressing. Jacks can be used effectively to reduce stresses in existing flanges. Cover plates are welded before removing jacks.

(4) Shear Reinforcement. Vertical stirrups serve as hangers which support the beam from the uncracked portion of concrete near the column.



(TYP.)
1. STRENGTHENING THE FLANGE
2. STRENGTHENING THE WEB
NEW FLANGE ANGLES ONE EACH SIDE OF WEB
REMOVE EXISTING STIFFENERS & REPLACE AS SHOWN
NEW STIFFENER SECTIONS
LIMITS OF CORROSION OF FLANGE
LENGTH AS REQ'D TO TRANSFER STRESS FROM EXISTING TO NEW FLANGE ANGLES
3. STRENGTHENING THE FLANGE WHERE IT IS NOT ACCESSIBLE FOR ADDING COVER PLATES
A. PLATING
REMOVE EXISTING CONCRETE SLAB
1. EXISTING CONDITION
NEW CONCRETE SLAB
DETAIL-A
2. ALTERED CONDITION
WELDED STUD SHEAR CONNECTORS
3"
DETAIL-A
B. INDUCE COMPOSITE ACTION
DEFLECTED TENSION TIE
THROUGH BOLTS
ANCHORAGE
CRACK
CONCRETE BEAM
END DIAPHRAGM & CONNECTION
STRINGER
1. BEFORE STRENGTHENING
TENSION CABLE
TURNBUCKLE
ANCHORAGE
STIFFENER
TENSION CABLE
TURNBUCKLE
2. AFTER STRENGTHENING
C. POST STRESSING
WELD COVER PLATE TO FLG. OF BEAM WHILE JACKING LOAD IS ACTING
TEMPORARY STIFFENERS AS REQUIRED
REACTION
JACKING LOAD
REACTION
STEEL STRINGER
D. PRESTRESSING
COLUMN
A
A
EXISTING CRACKS
ELEVATION
NEW REINFORCMENT
NEW CONCRETE ENCASEMENT

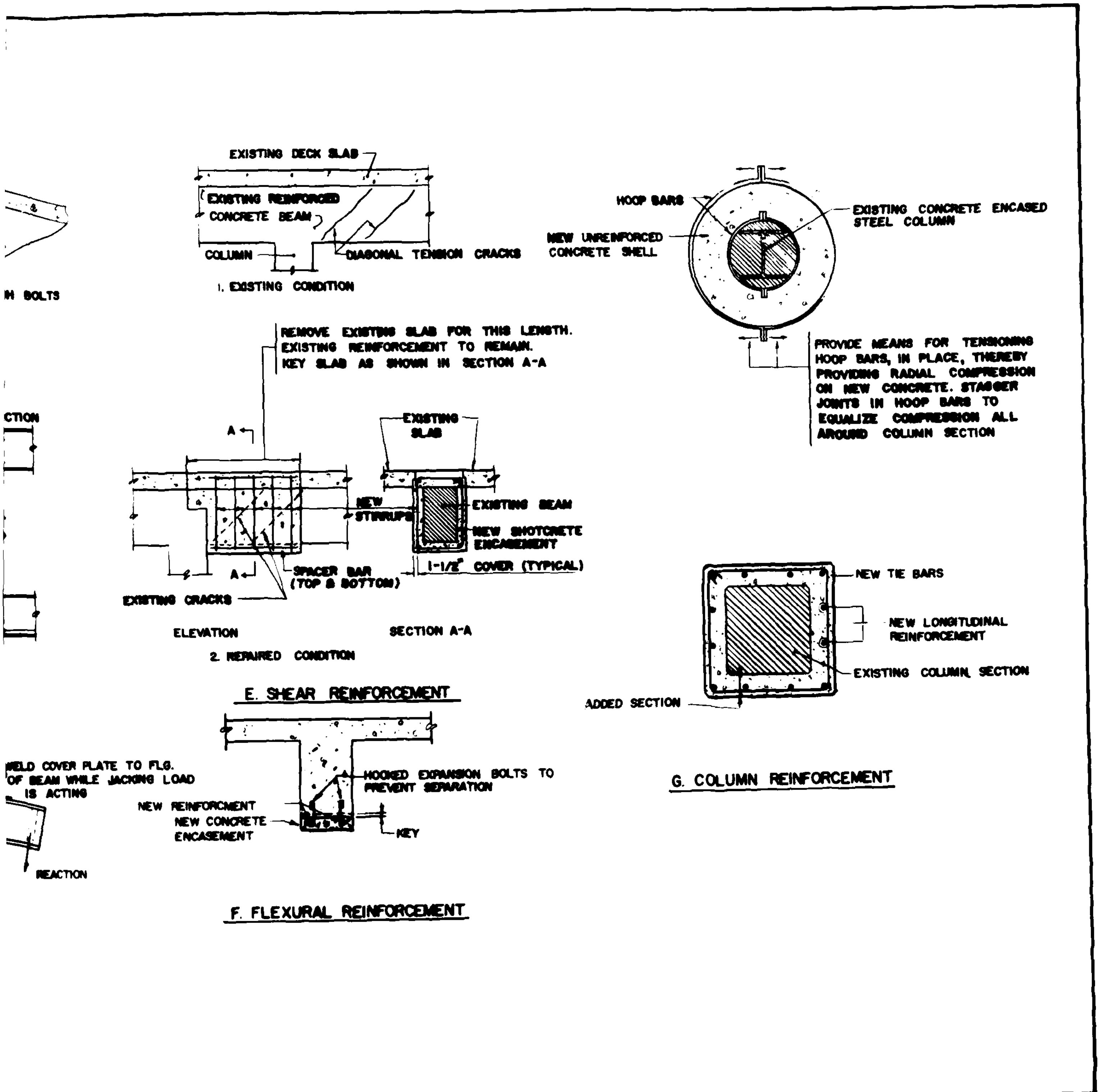


FIGURE 18
Methods of Strengthening
An Existing Structure
25.6-49

2

(5) Flexural Reinforcement. Longitudinal reinforcement can be added effectively if positive means for preventing separation and for transferring horizontal shear are used.

(6) Column Reinforcement. Column sections may be strengthened by adding concrete with longitudinal and lateral reinforcement or by adding unreinforced concrete restrained by hoop bars.

b. Compatibility. Details must consider any inherent incompatibility of old and new materials. Provision must be made to resist separation forces. New concrete will have a different modulus of elasticity, coefficient of thermal expansion, and shrinkage than old concrete. Consider differing expansion effects due to differing absorption of moisture. Provide resistance against "curling" due to thermal gradients. Compatibility of connectors must be considered. For example, rivets or bolts are not compatible with welds. Friction bolts are not compatible with rivets. Creep is an important factor.

c. Dead Load vs. Live Load Stresses. Unless the load on a structure is relieved (for example, by removal or by jacking), the existing framing will continue to carry: (1) the full dead load of the construction, (2) any part of the live load which is in place when the new framing is connected, and (3) a proportionate share of the live load subsequently added. The new framing will carry only a part of the live load. As a result, under the final loading condition, the stresses in the new and existing material of the same or similar members will be different and often radically so. For example, assuming a 1:1 ratio of dead to live load and of new to existing material in the cross section of a given member, and setting aside considerations of plastic deformation, the stress in the existing material would be three times the stress in the new. As a result, the new material cannot be stressed up to allowable values without simultaneously overstressing the existing sections. It is necessary either to provide an excess of new material or to relieve the load on the structure before strengthening.

d. Exception. The discussion of paragraph c above may not apply if plastic deformation of the structure (and its associated, increased deflection) can be permitted. For example, as regards flexural members, in general, the plastic hinge moment capacity is not reduced by locked-in stresses, but for compression members, locked-in stresses reduce the buckling strength. The magnitude of the reduction is a function of the L/r value as shown in Figure 19.

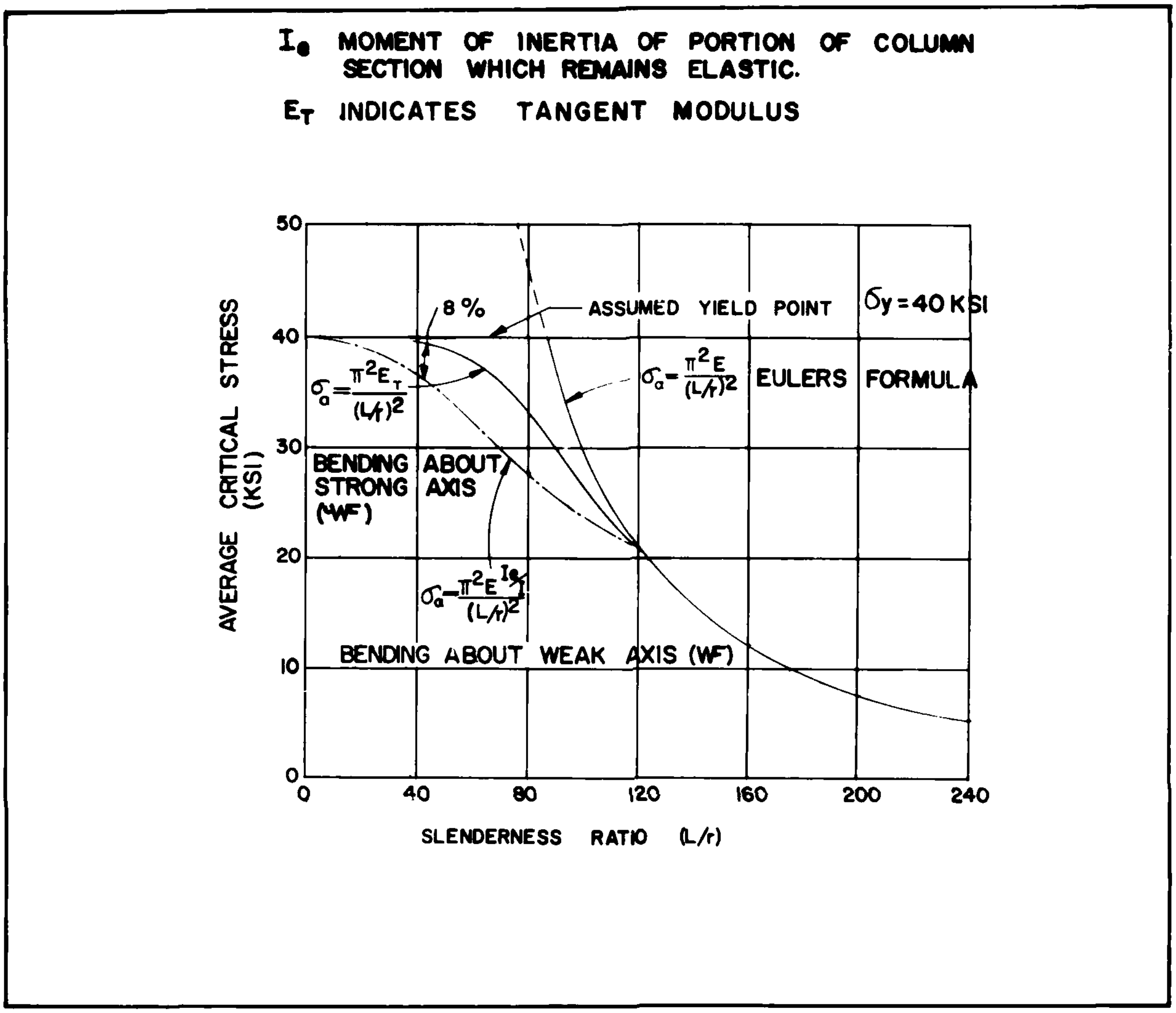


FIGURE 19
Effects of Residual Stress on Column Strength

Section 7. DETERIORATION OF WATERFRONT STRUCTURES.

1. CAUSES. The more common causes of deterioration associated with waterfront structures are:

a. Steel Structures.

(1) Corrosion (See Figures 20 through 23).

(2) Abrasion (See Figure 24).

(3) Impact (See Figures 25 and 26).

b. Concrete Structures.

(1) Corrosion of Reinforcement (See Figure 27).

(2) Chemical Reactions (See Figures 28, 29, and 30).

(3) Weathering (See Figure 31).

(4) Swelling of Concrete (See Figure 32).

(5) Impact (See Figures 33, 34 and 35).

c. Timber Structures.

(1) Decay (See Figures 36, 37, and 38).

(2) Borer Attack (See Figure 39).

(3) Corrosion and Abrasion of Hardware (See Figures 40 and 41).

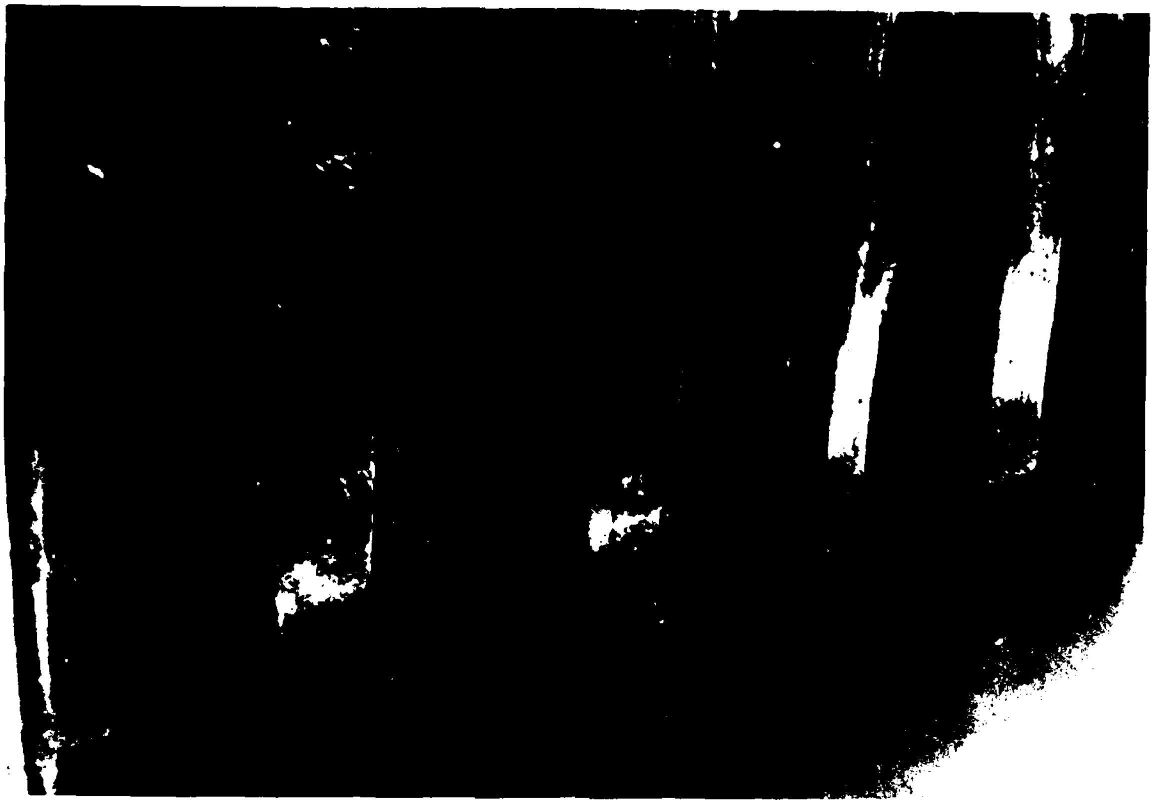
(4) Impact (See Figure 42).

2. PREVENTIVE MEASURES IN DESIGN AND CONSTRUCTION.

a. Steel Structures.

(1) General. All parts which will be subject to corrosion must be accessible for inspection and repair. If not accessible, encase with concrete or provide some other long-life, high-resistance type of coating.

(2) Shapes. Select shapes which have a minimum of exposed surface. For example, use T's instead of double angles.



Note holes at and below low water level, and in splash zone.
These are the zones of maximum attack.

FIGURE 20
Corrosion of Steel Sheet Piling

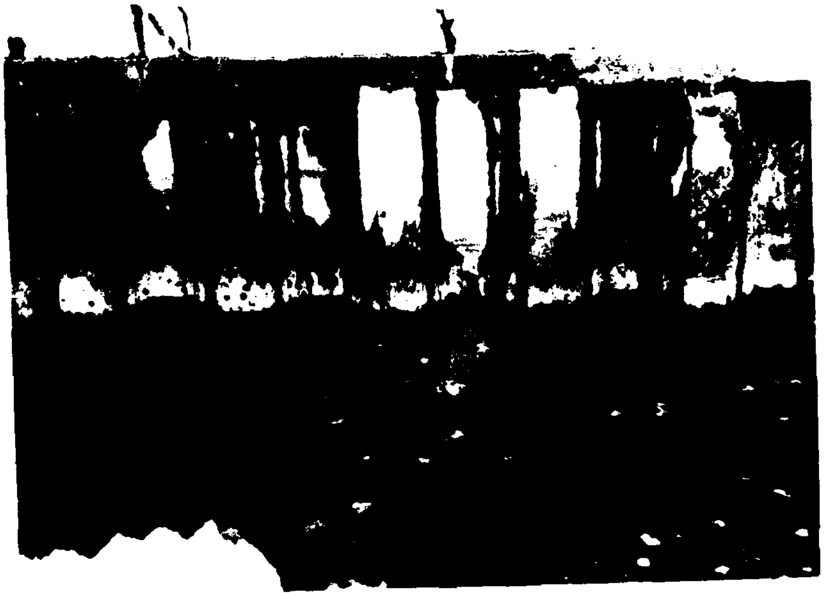


FIGURE 21
An Advanced Case of
Corrosion of Steel Sheet Piling

a b

Figure a shows the appearance of the beam before removal of the rust scale. Except for some pitting and tuberculations, it looks relatively sound. Figure b shows the same view after the rust scale had been partly removed by hammering. Note that the entire appearance has changed, revealing a very serious condition of corrosion. The flange has been reduced to sheet metal, with holes indicated by the arrows. These photographs illustrate the fact that the severity of corrosion in a steel member should not be judged until after the rust scale has been removed.

FIGURE 22
Corrosion of Low Water Brace Beam

FIGURE 23
Corrosion of Steel H-Piles

FIGURE 24
Result of Abrasion of
Steel Sheet Piling in Surface Zone

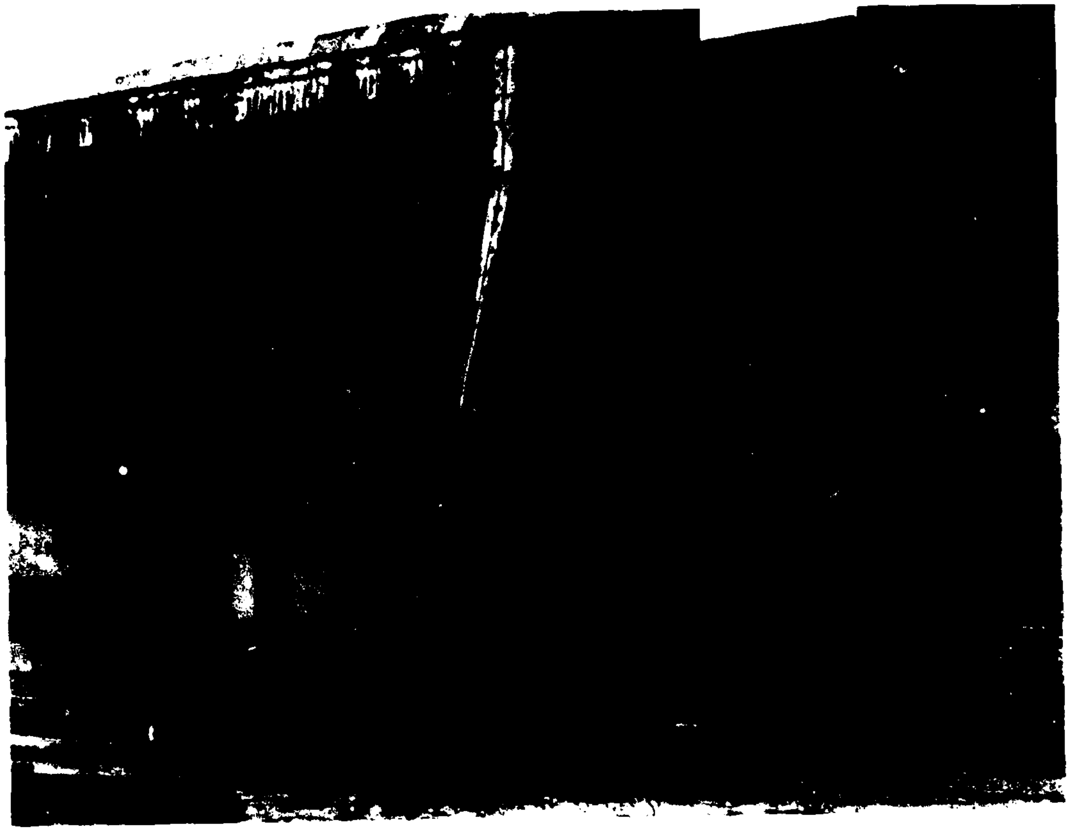
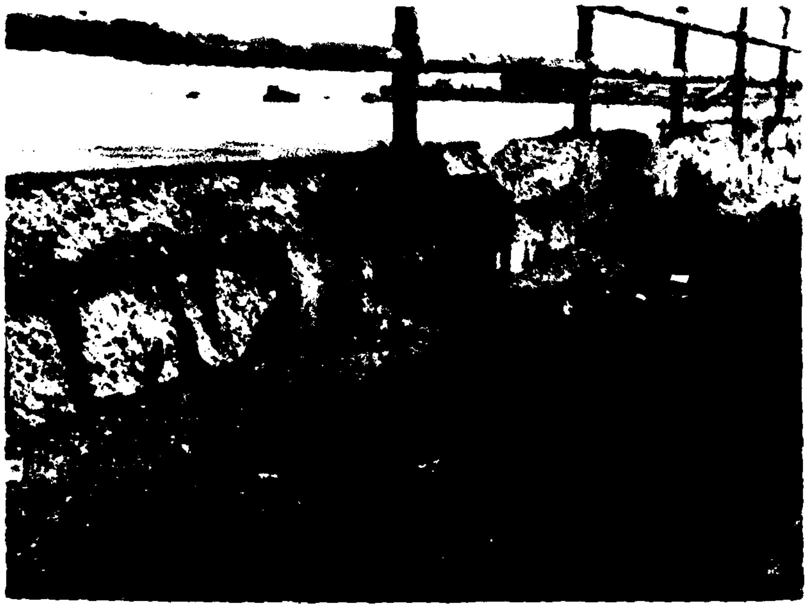


FIGURE 25
Impact Damage Due to Berthing of Ships

FIGURE 26
Impact Damage Due to Berthing of Ships

Spalling of Soffit of Slab

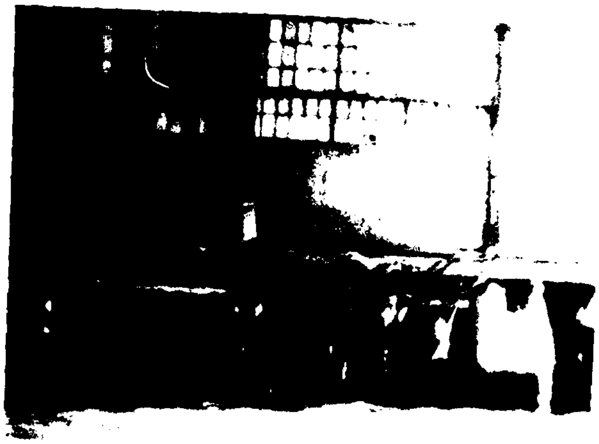


Spalling of Curb of Wharf

FIGURE 27
Spalling of Concrete
Due to Corrosion of Reinforcement

Note protusion of aggregate particles from cement-sand matrix; abrupt limit of deterioration near high water mark (black horizontal band on the pile); and spalling of corners due to corrosion of corner bars.

FIGURE 28
View of Concrete Pile
Deteriorated by Seawater - Sulphate Attack
25.6-61



As-built Condition

Initial Phase of Attack

Advanced Deterioration

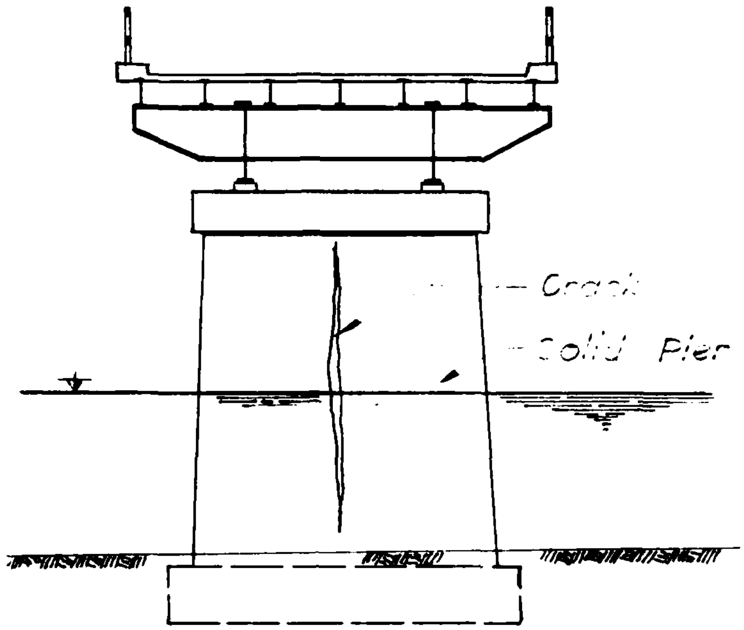
FIGURE 29
Spalling of Mass Concrete
Due to Freeze-Thaw and Sulphate Attack
25.6-62

This is Dock No. 6 at Brooklyn. Most of the tremie docks built at this time show similar conditions.

FIGURE 30
View of Interior of Drydock
Where Walls Have Undergone Internal Expansion
Due to Reaction of Sulphates in Seawater
with Cement in Concrete

Deterioration of precast concrete piling in a marine environment. Observe the condition of the piles in the rear row of verticals. The condition shown was attributed to weathering plus abrasion by floating ice. The reinforcing bars are still in relatively good condition. The structure was about 17 years old when this photograph was taken. Compare this photo with Figure 46.

FIGURE 31
Spalling of Concrete Due to Weathering
(Freeze-Thaw)



This pier was built in a cofferdam. Concrete was placed "in-the-dry". After removal of the cofferdam, the dry concrete became saturated and swelled. Above the saturation line the concrete did not swell. Cracking resulted from the differential expansion. Heavy reinforcement in the cap section prevented the crack from propagating to the top of the pier.

FIGURE 32
Swelling of Concrete Due to Absorption of Water

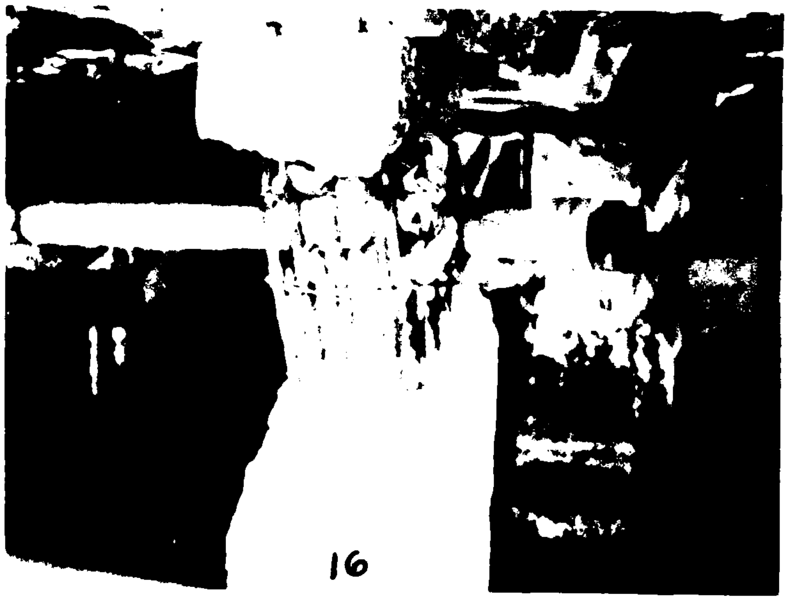


FIGURE 33
Damage to Concrete Pile Due to Berthing Impact

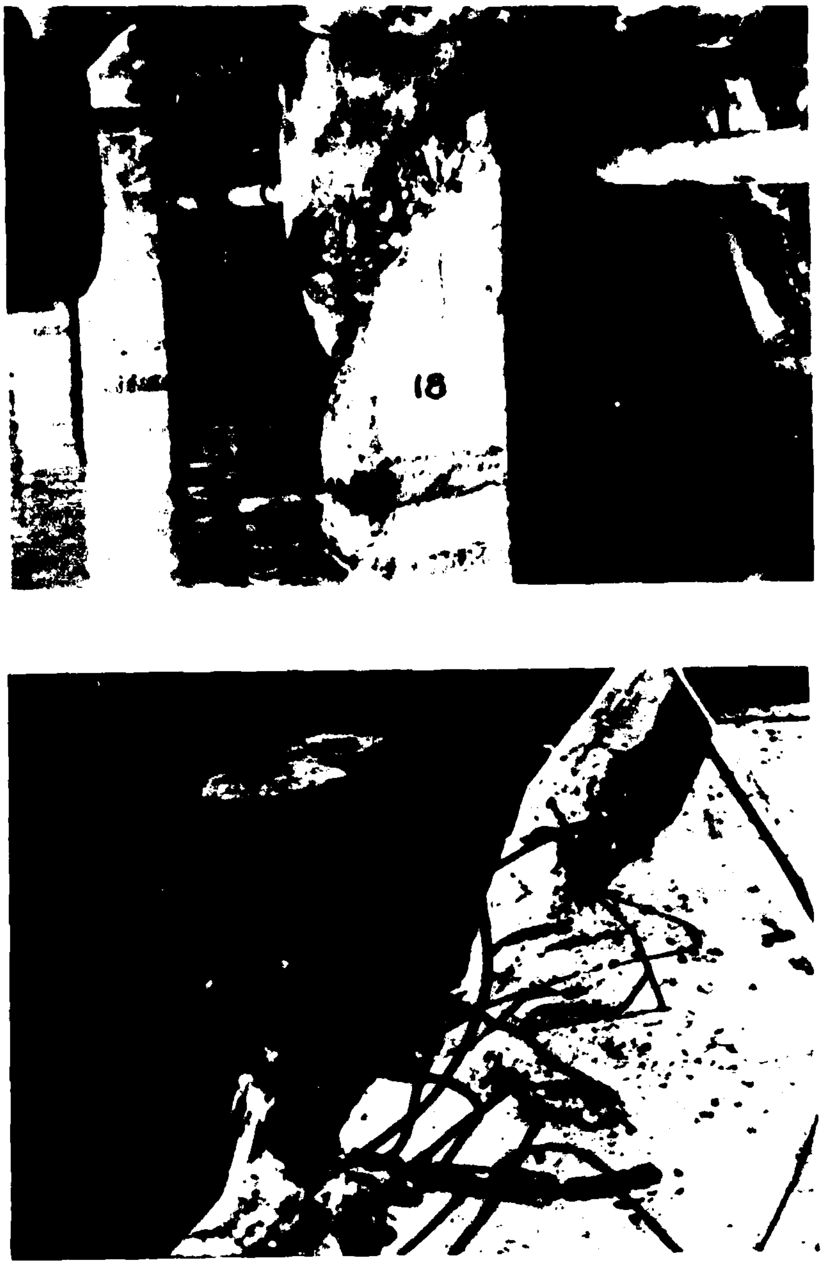
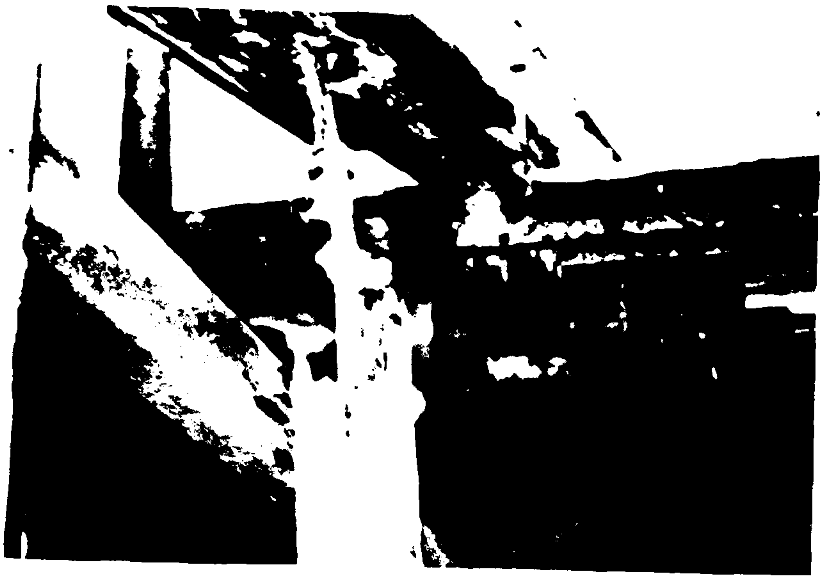


FIGURE 34
Impact Damage Due to Berthing of Ships

25.6-67

FIGURE 35
Impact Damage

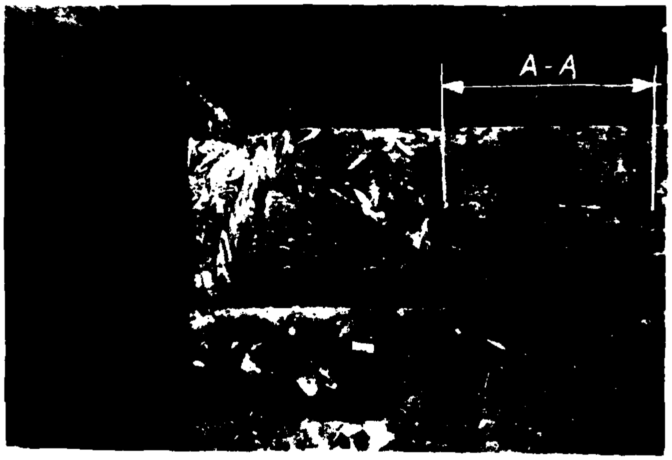
FIGURE 36
Decay of Timber Piles
25.6-69



Advanced Decay of a Timber Pile

Culmination of Neglect of Decay in Timber Supports

FIGURE 37
Decay of Timber Structures
25.6-70

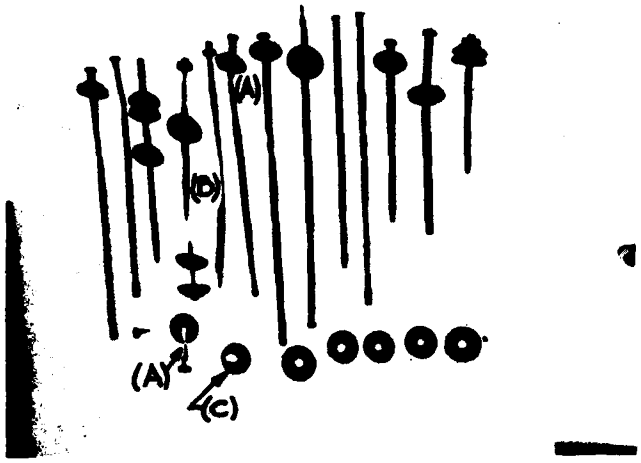


Before the chock was cut, the entire piece looked like the section marked A-A. Note the rot between the timbers revealed by making the cut. This is a typical situation, and illustrates the need to look closely and deeply for evidence of deterioration and the need to know where to look.

FIGURE 38
Checking Timbers for Rot

Timber Attached by Limnoria

FIGURE 39
Borer Attack on Timber
25.6-72



Note "ne.‹ing-down" of bolts and that "necking-down" is localized where the bolt shank was exposed: (1) next to the washers (A), and (2) between adjacent timbers (B). Note also, the distortion and enlargement of the holes in the washers (C). Observe also that some of the bolts have no heads. They were corroded away. Similarly, no nuts were found. Presumedly, they too were corroded away. The bolts were held in place by friction until lateral forces due to waves and ice pulled them loose, at which point bolts and timbers were lost.

FIGURE 40
Samples of Hardware Taken From Pier Structure

Note distortion and enlargement of holes in washers. Corrosion of bolt heads and nuts loosened the connections. The resulting working of the timbers caused the bolt shank to "saw" on the washers, wearing both shank and washer. After a while, the hole in the washer was enlarged and the size of the bolt head was reduced (by corrosion), permitting the bolt head to be drawn through the hole in the washer, thus freeing the connection.

FIGURE 41
Detail of Washers Shown in Figure 40

FIGURE 42
Impact Damage to Fender System

(3) Detailing. Detail so that accumulations of dirt and debris will be avoided. Avoid narrow crevices which cannot be painted or sealed. Draw faying surfaces into tight contact by use of closely spaced stitch rivets, bolts, or welds. Prime faying surfaces before assembly.

(4) Minimum Thickness of Metal. For piling deck and substructure framing and bracing, and hardware and fittings, see Section 2, 3 and 4 respectively. For other framing, no minimum thickness requirement is established.

(5) Drainage. In general, detailing framing to shed water is the single most important factor in inhibiting corrosion and deterioration of coatings (except the need for good workmanship). If the potential for ponding is unavoidable, provide drain holes. Drain holes should be a minimum of 4-inch diameter to inhibit clogging.

(6) Sacrificial Metal. Use is discouraged in favor of the use of protective coatings.

(7) Cathodic Protection. (See Section 5, paragraph 3.)

b. Concrete Structures.

(1) Class of Concrete. (See Sections 2 and 3.)

(2) Cover Over Reinforcement. (See Sections 2 and 3.)

(3) Quality of Concrete. Good quality is the important factor in obtaining a dense concrete. This, in turn, is the most important factor in preventing penetration of moisture, which is the primary cause of deterioration of concrete. Do not use poorly graded aggregate, or a water-cement ratio greater than 6 gals/sack of cement, reduced to 5 gals/sack of cement for thinner sections such as slabs and wherever clear cover over reinforcement is 2 inches or less. Watertight concrete can be obtained by using air entrainment (maximum 6 percent by volume) and a water-cement ratio not greater than 5 gals/sack of cement.

(4) Types of Concrete. Never use Type III (high early strength) cement and, in general, avoid the use of Type I cement in a salt water environment. Use Type II (sulphate-resistant). Type III cement is excessively susceptible to sulphate attack. Use of Type V (high sulphate-resistant) cement seldom is required.

(5) Expansion Joints. Provide an adequate number of expansion joints. Use types of expansion joints such as double bents with movement taken up by bending of the piles or elastomeric pads (See Figure 43) with some form of joint sealer. Types of expansion joints to be avoided are shown in Figure 44. Experience indicates that such joints do not give good service.

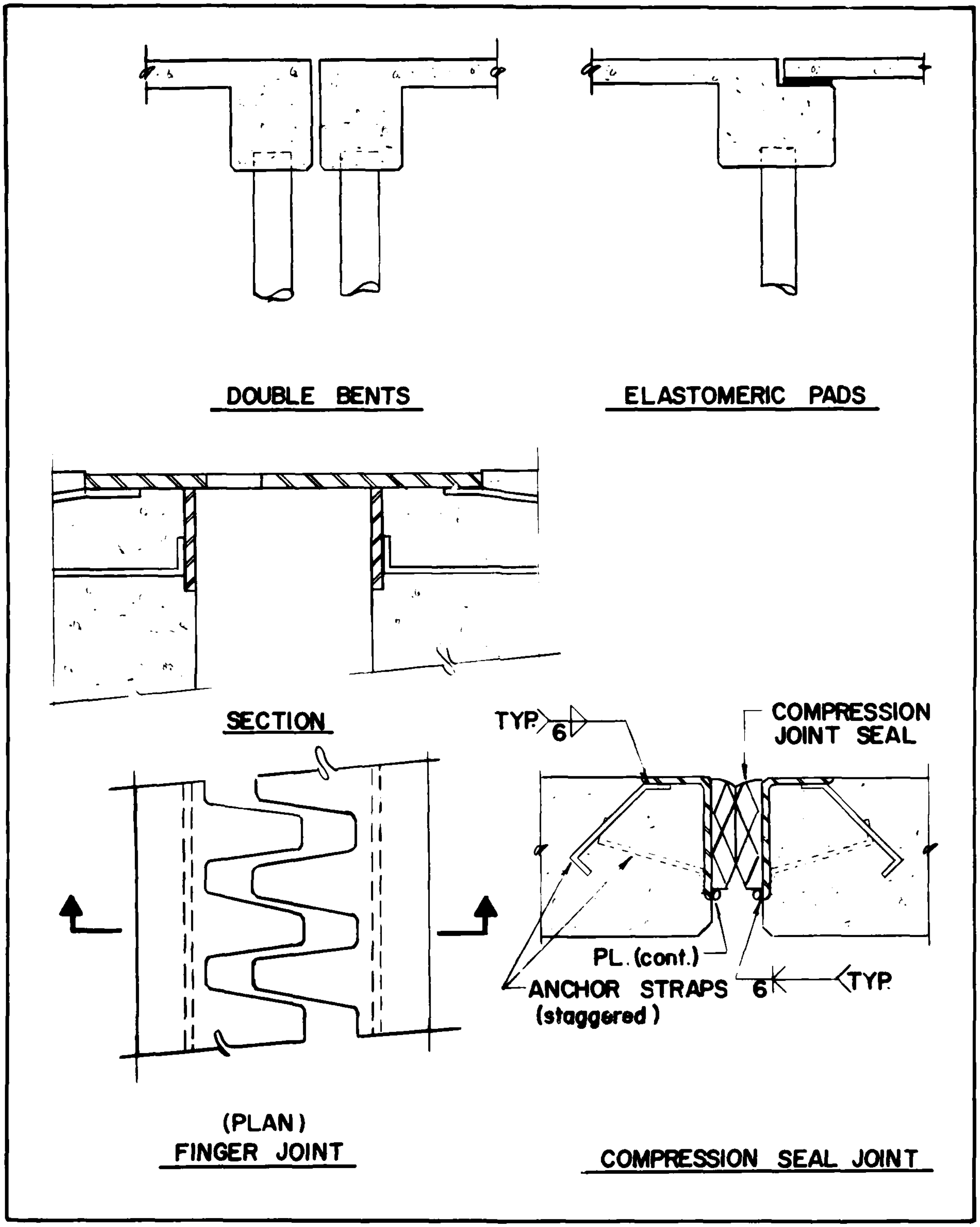


FIGURE 43
Recommended Types of Expansion Joints

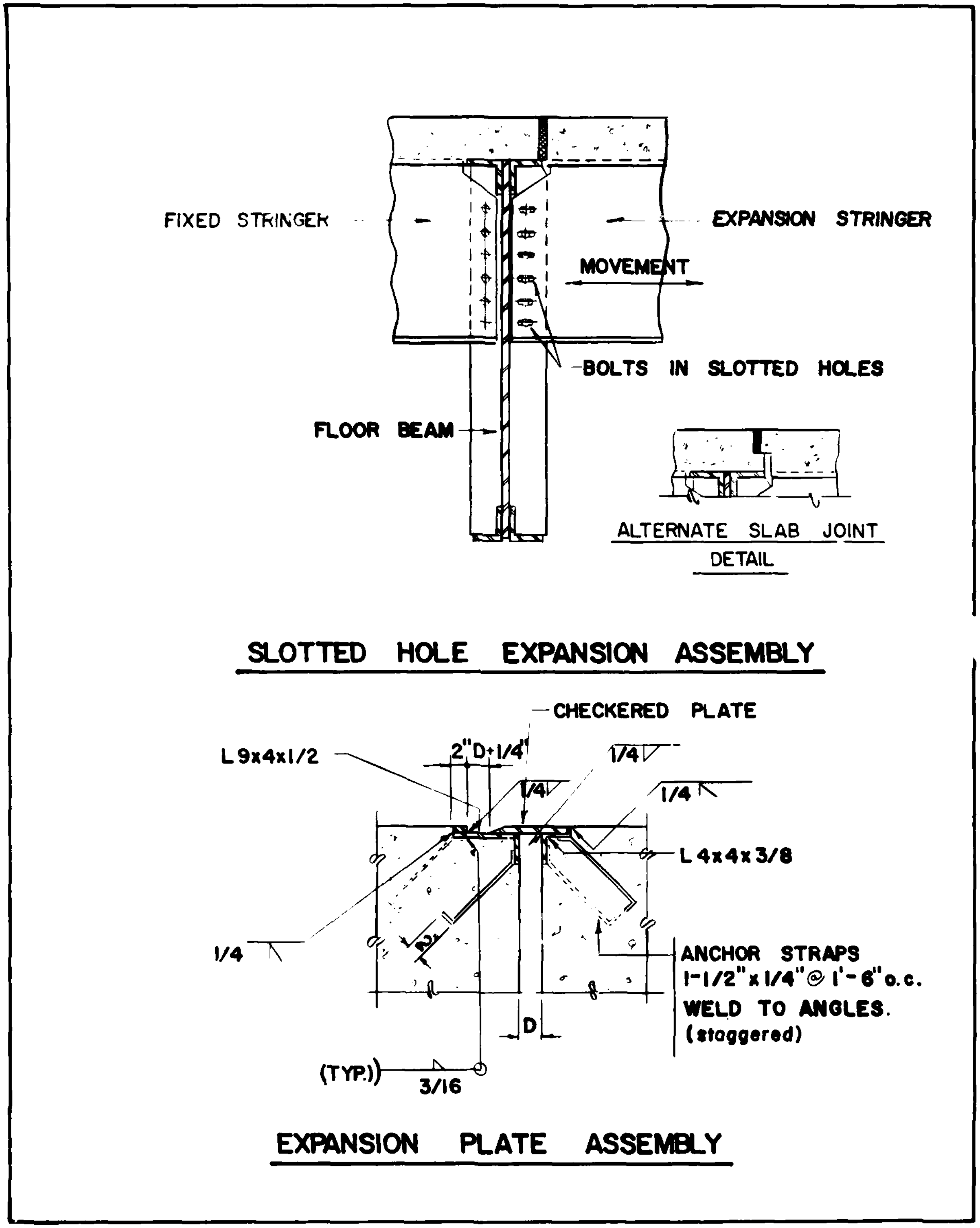


FIGURE 44
Types of Expansion Joints Not Recommended

(6) Tropical Climates. In tropical climates, in areas subject to salt spray, consider the use of galvanized or plastic coated reinforcing bars. If plastic coated bars are used, pay special attention to bond stresses.

(7) Mix. Avoid excessively rich mixes (over 6 bags/c.y.) as excess cement tends to enhance the potential chemical reaction with seawater.

(8) Jackets and Facings. Timber jackets for concrete piles (see Figure 45) and stone facing for concrete seawalls work extremely well to prevent deterioration due to corrosion of reinforcement, weathering, and chemical attack. They tend to isolate the concrete from chemical constituents in the environment, insulate against freezing, and keep free oxygen from the reinforcing bars. See Figures 46 and 32 for comparison of precast concrete piling installed with and without timber jackets.

(9) Aggregate. For most aggregates, alkali-aggregate reaction can be prevented by specifying maximum alkali content of the cement (percent Na_2O, plus 0.658 times percent K_2O) not to exceed 0.60 percent.

(10) Surf Zone. In a surf zone, increase the concrete cover and streamline sections to prevent abrasion. The best solution, if economically feasible, is a granite or other hard facing.

(11) Additives. Calcium chloride (as an accelerator) shall not be used in prestressed concrete and concrete exposed to seawater. The use of calcium nitrite or other chemicals as a deterrent to corrosion of embedded reinforcing steel is not an adequate substitute for good quality concrete and adequate cover. In extreme cases, use of coated reinforcing bars may be required.

(12) Drainage. Where feasible, detail scuppers and weep holes to drip clear of the underlying structure. Provide drip grooves in fascia beams and slab soffits.

(13) Compatibility of Sections and Materials. (See Figures 47 and 48 for some common problems.)

c. Timber Structures.

(1) Detail to minimize cutting, especially that which must be done after treatment.

(2) Detail to provide ventilation around timbers. Avoid multiple layers of timbers as decay is enhanced by moist conditions at facing surfaces. Curb logs should be set up on blocks. Walers should be blocked out from face of pier. Provide thin spacers between chocks and wales. Provide gaps between deck and tread planks.

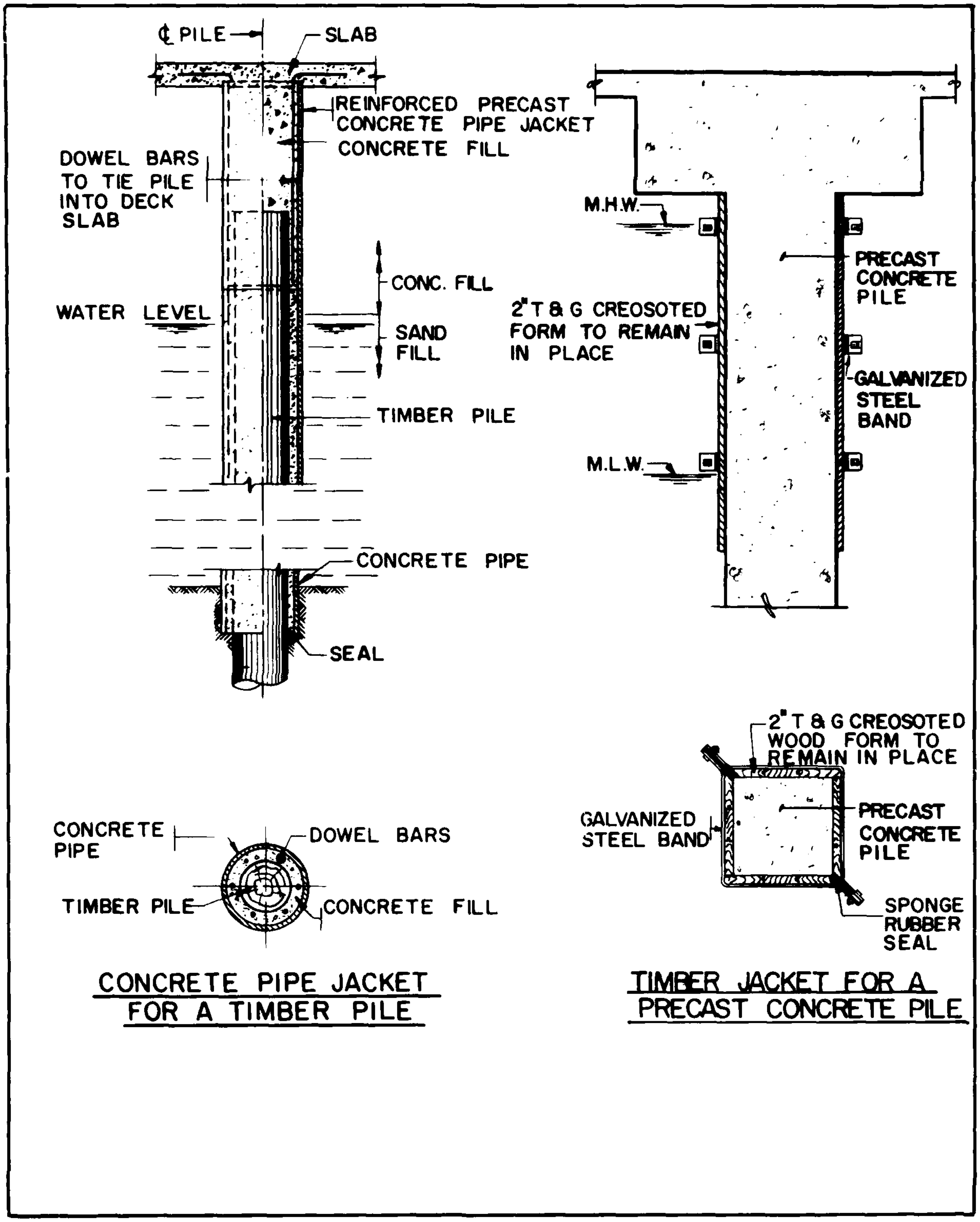
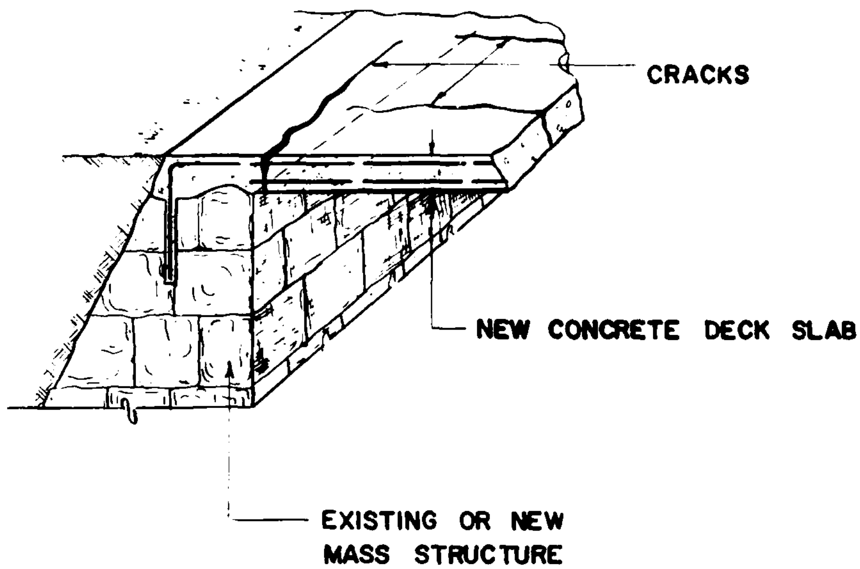


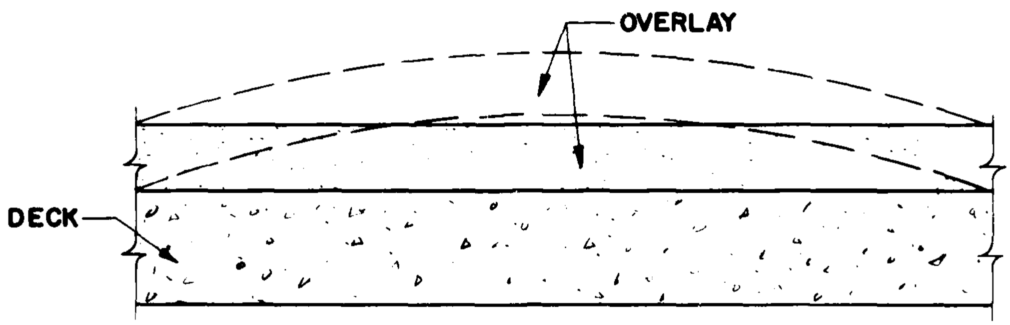
FIGURE 45
Pile Jackets

Timber jackets placed on new precast concrete piles. The piles shown in this photograph are about 25 years old and are located in the same waterway about 2 miles from those shown in Figure 31. Compare the condition. These piles are in excellent shape. This is the result of the use of timber jackets which were installed at the time that the trestle was built. Note that some of the bands have corroded through and the jackets have fallen off, but that the concrete, including that in the tidal range, is still in excellent condition.

FIGURE 46
Preventing Deterioration of Concrete in Marine Environment by the Use of Timber Jackets



CRACKING DUE TO AN ABRUPT CHANGE IN SECTION OF A CONCRETE STRUCTURE



BUCKLING OF APPLIED OVERLAY DUE TO A DIFFERENCE IN MATERIAL PROPERTIES

FIGURE 47

Compatibility of Sections and Materials

25.6-82

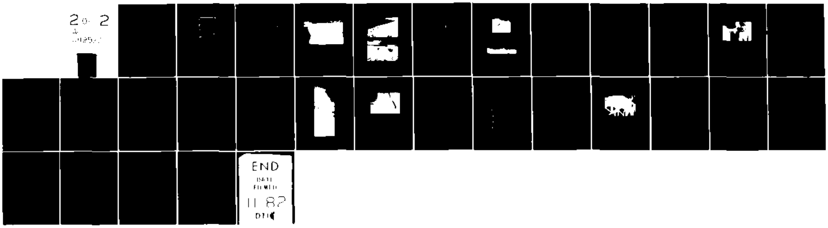
Note: Proper detailing would provide expa[illegible]
and mesh reinforcement.

[illegible]

Spalling [illegible]

Due t[illegible]

AD-A119 527 NAVAL FACILITIES ENGINEERING COMMAND ALEXANDRIA VA F/G 13/13
GENERAL CRITERIA FOR WATERFRONT CONSTRUCTION. DESIGN MANUAL 25.--ETC(U)
JUL 81
UNCLASSIFIED NAVFAC-DM-25.6 NL



(3) For requirements concerning hardware, preservative treatment of the wood, protection of tops of piles, and selection of species of wood, see Sections 2 and 3.

3. SOME CASE HISTORIES.

CASE HISTORY NO. 1

a. The Problem. Existing sheet pile wall damaged by corrosion and berthing impacts.

b. The Solution. (See Figure 49.) New bulkhead wall, with existing wall acting as anchorage (bin structure). Construction confined to area outboard of existing wall in order to avoid disturbance of maze of existing utilities.

c. Note. After about 15 years of service, failures began to occur due to the "U"-bolts (Plan Section A-A of Figure 49) tearing the interlocks to which they were anchored. Up to 6 inches of deflection of the top of the outboard sheet piles had developed. The condition was then repaired as shown in Figure 50. See Figure 51 for a view of the final construction.

CASE HISTORY NO. 2

a. The Problem. (See Figure 52.) Existing sheet pile wall severly damaged by corrosion.

b. The Solution. (See Figure 53.) Existing sheet piling "preserved" by external facing. Corrosion loss due to attack from inside face assumed to be negligibly small. Strength of existing wall checked as indicated in Figure 17.

CASE HISTORY NO. 3

a. The Problem. (See Figure 54.) Sulphate attack caused disintegration of concrete seawall.

b. The Solution. (See Figure 55.) Contact between existing concrete and seawater (reacting ingredients) cut off by interposing layer of sulphate-resistant concrete. Note that all of the old concrete was not replaced -- only what had been severely attacked. Concept is to inhibit further attack by isolating concrete from aggressive environment.

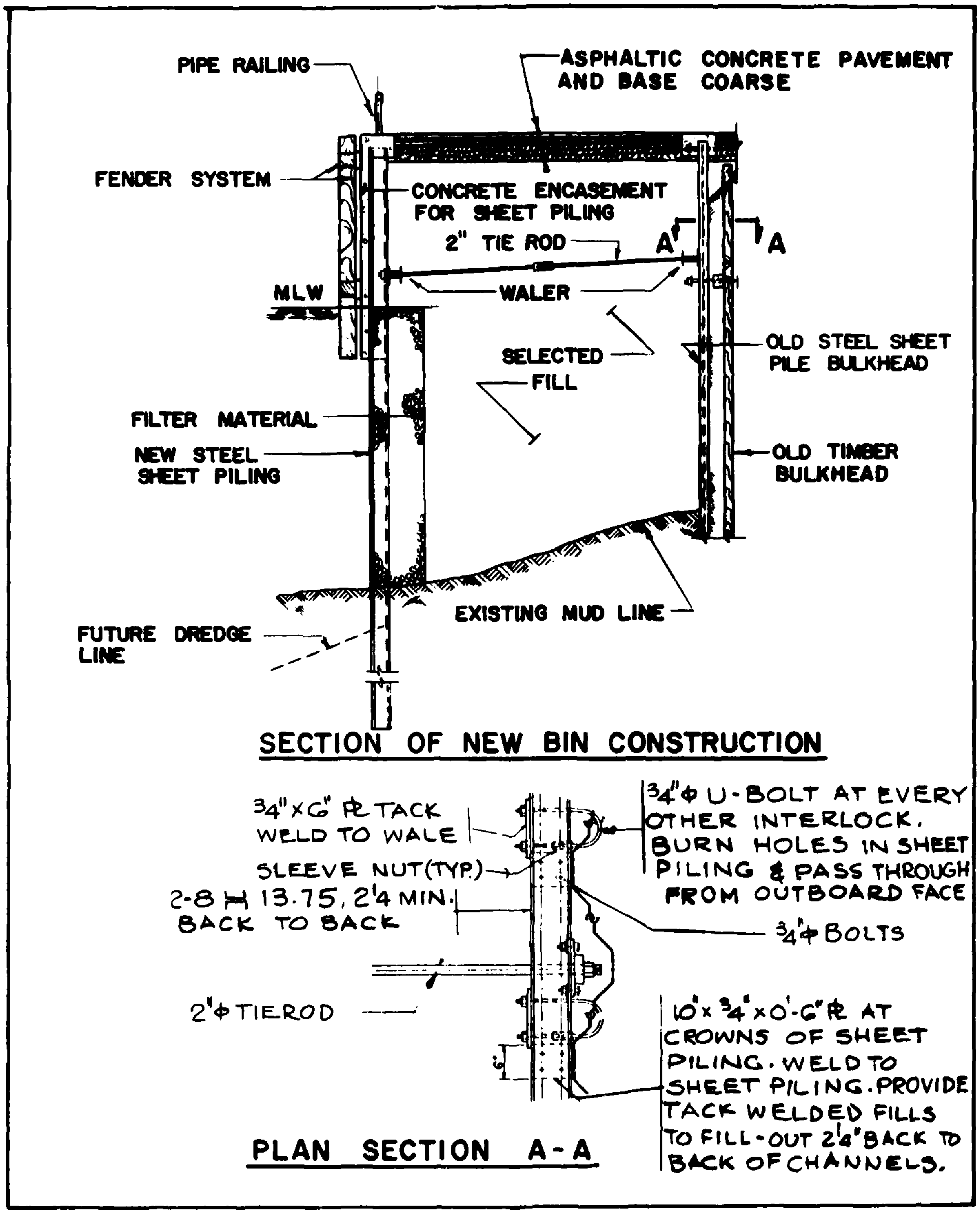


FIGURE 49

Case History No. 1 -
First Repair

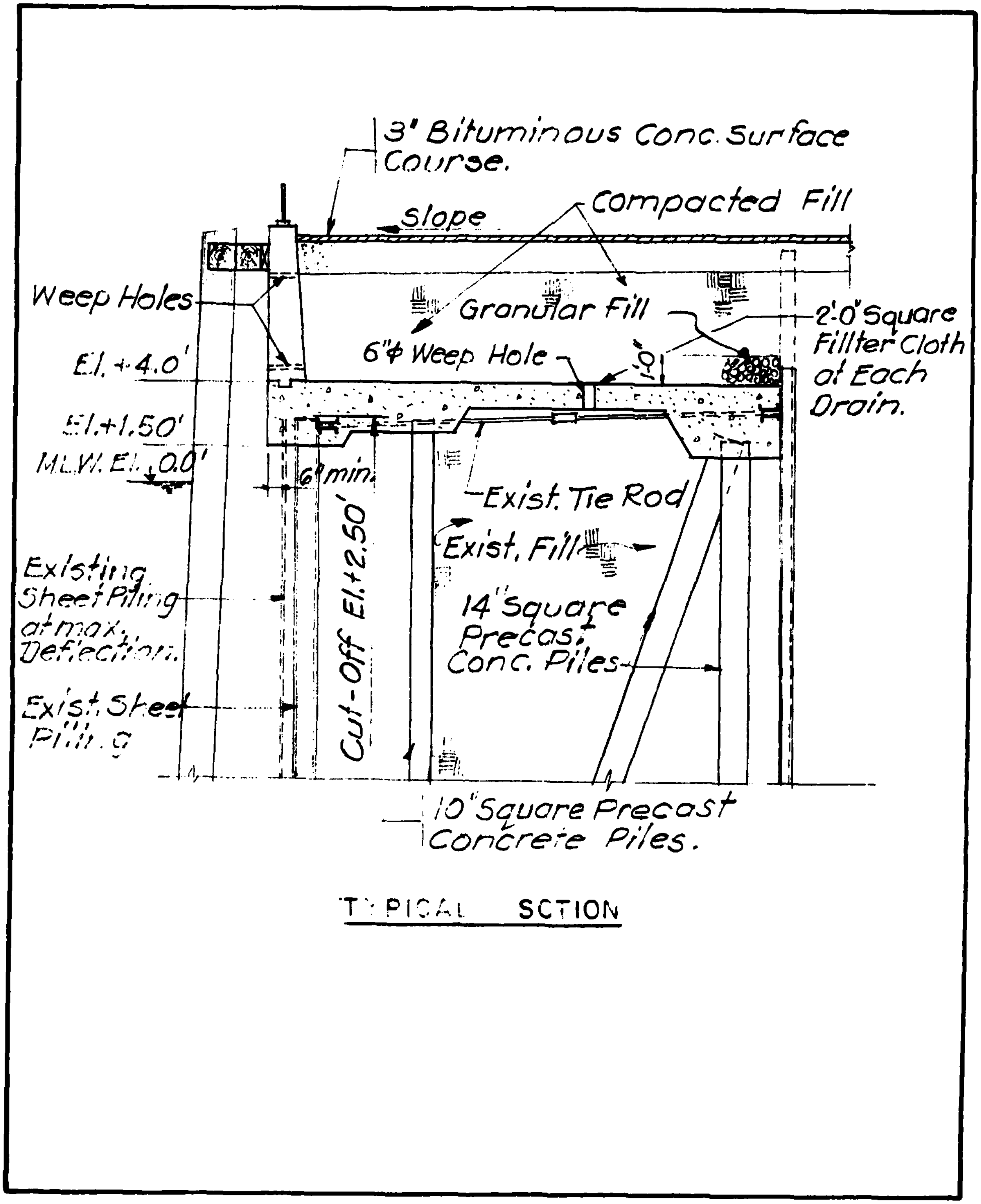


FIGURE 50
Case History No. 1 -
Second Repair

FIGURE 51
Case History No. 1 - Final Construction

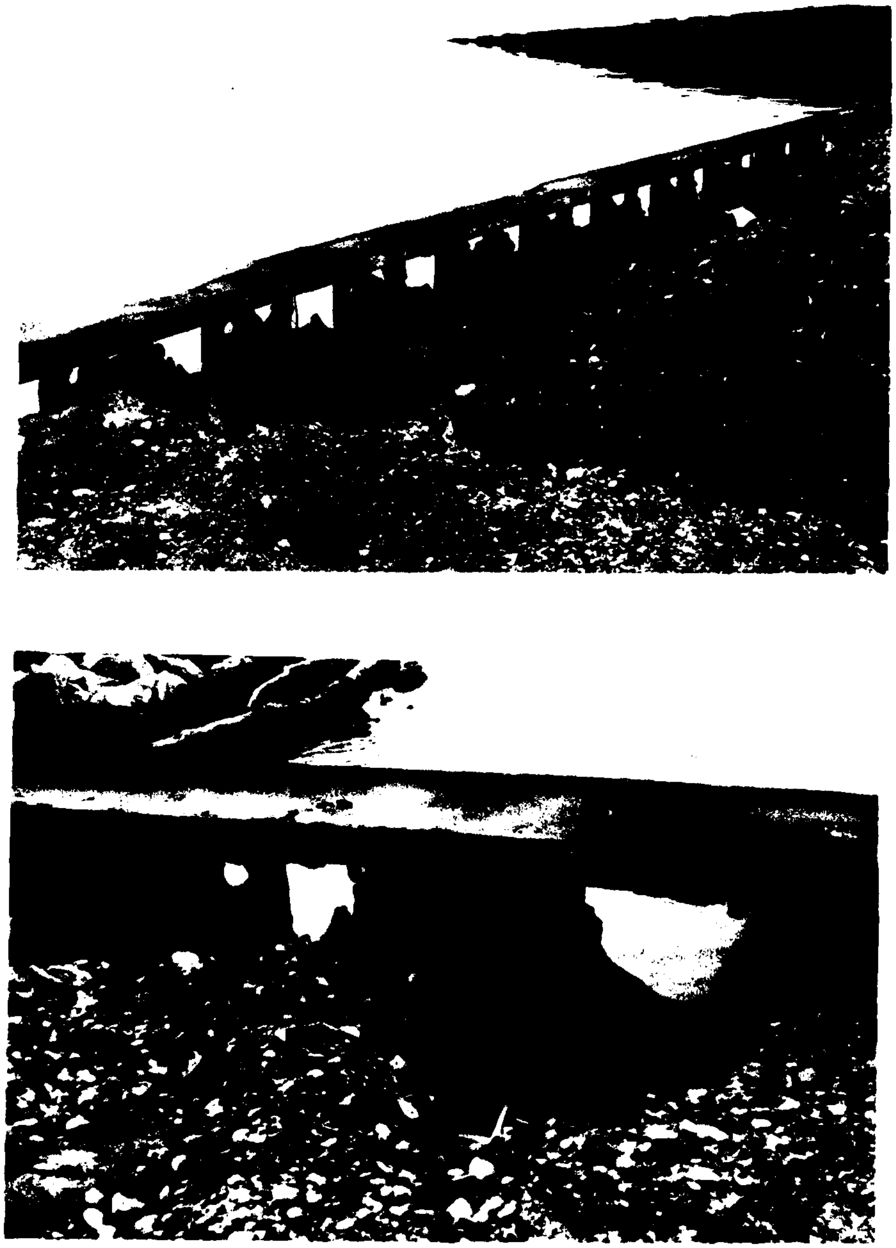


FIGURE 52
Case History No. 2 -
Deterioration of Top of Bulkhead
25.6-88

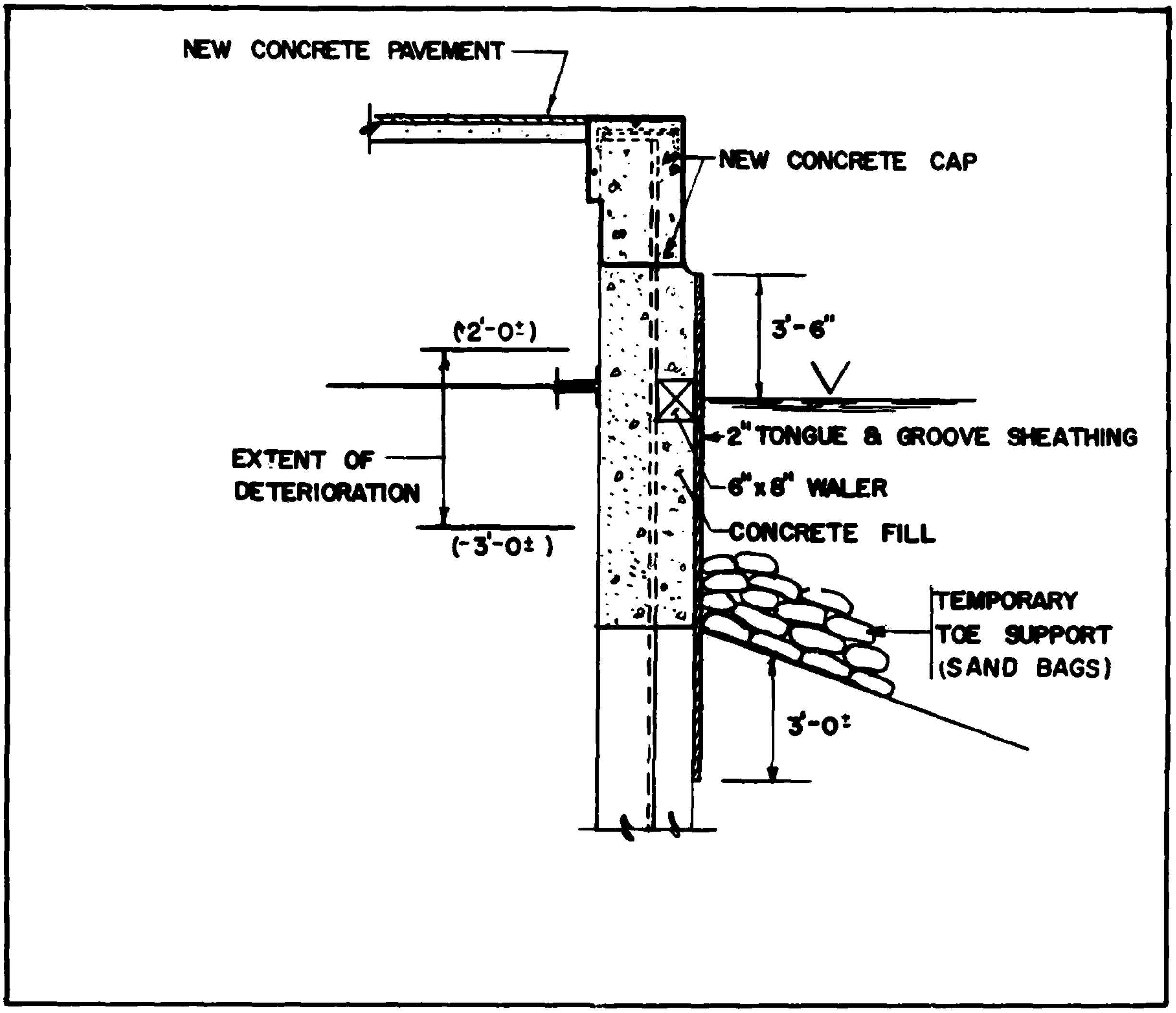
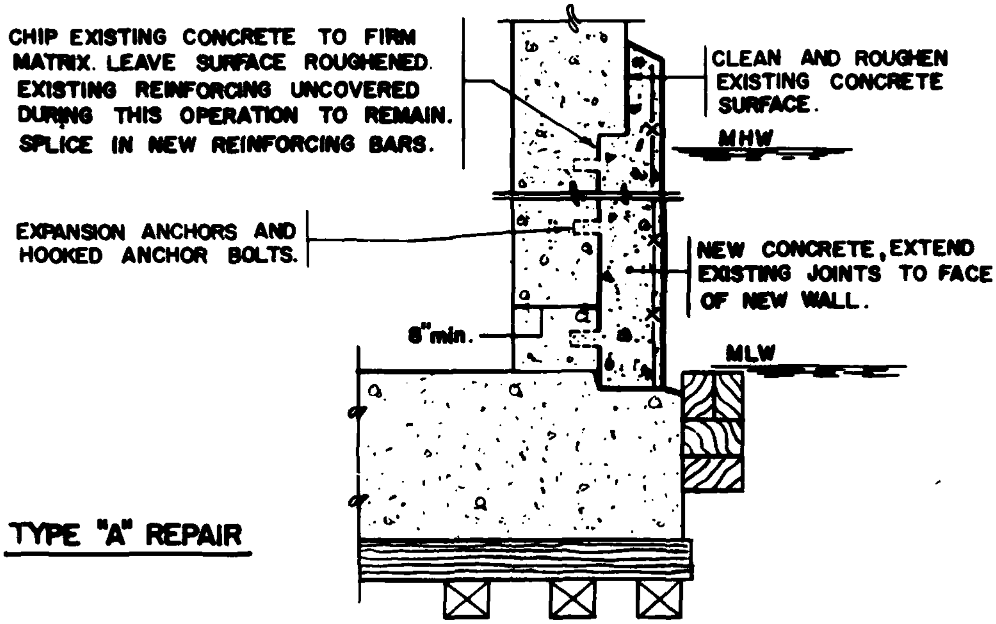


FIGURE 53
Case History No. 2 -
Repair Details

FIGURE 54
Case History No. 3
Desintegration of Concrete Seawall by Sulphate Attack
25.6-90



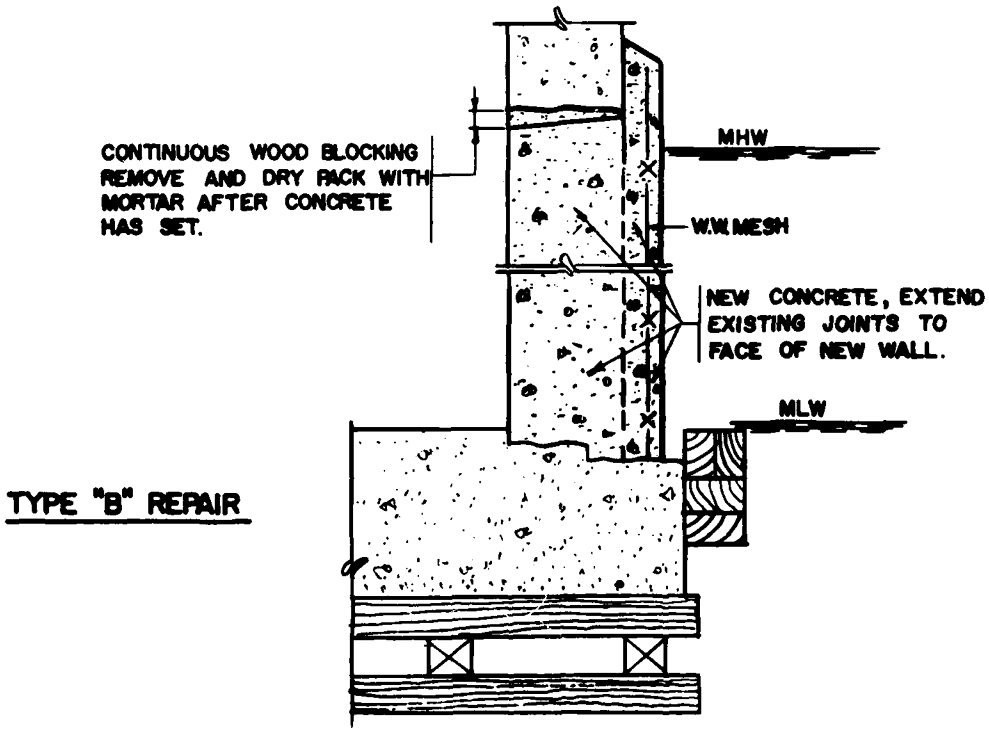


FIGURE 55

Case History No. 3 -
Repair Details

CASE HISTORY NO. 4

a. The Problem. (See Figure 23.) Corrosion of steel H-piles.

b. The Solution. (See Figure 56.) Corrosion was concentrated in a narrow band near and below mean low water. This is a typical condition. Above and below this band, sufficient section remained in piling that projecting at an estimated rate of loss based on observed rate of loss, 25 years of service life remained before critical stress levels (defined as load factor of 1.33 on dead plus live load) would be reached.

CASE HISTORY NO. 5

a. The Problem. Wood pier deteriorated as shown in Figure 37. Deterioration was due to rot. See Figure 57 for typical pile condition. Below mid-tide level piles were sound. Deck timbers were creosoted and piles were untreated. No borer activity due to polluted harbor waters.

b. The Solution. Remove the superstructure and deck. Post existing piles above mid-tide level. (See Figure 58.) New deck on posted piles. Salvage of lower portions of existing piles (and of some of the cross caps) reduced repair cost by an estimated 30 to 40 percent.

CASE HISTORY NO. 6

a. The Problem. Deterioration of precast concrete piles due to sulphate attack. (See Figure 28.) Piles were made using Type III, (high early strength) cement. Attach extended to mud line.

b. The Solution. Jacket with concrete. (See Figure 59.)

CASE HISTORY NO. 7

a. The Problem. Attack by limnoria (lignorum) and pholadidae (pill bugs) of creosoted timbers and piles (location - Bermuda). (See Figure 60 for cross section of structure.)

b. The Solution. Wrap piles with plastic sheet. Check design, and upon determining that it was structurally feasible, raise pile bracing above water. (See Figure 61.)

CASE HISTORY NO. 8

a. The Problem. Corrosion of steel H-piling. (See Figure 12 for a typical cross section of the pier.)

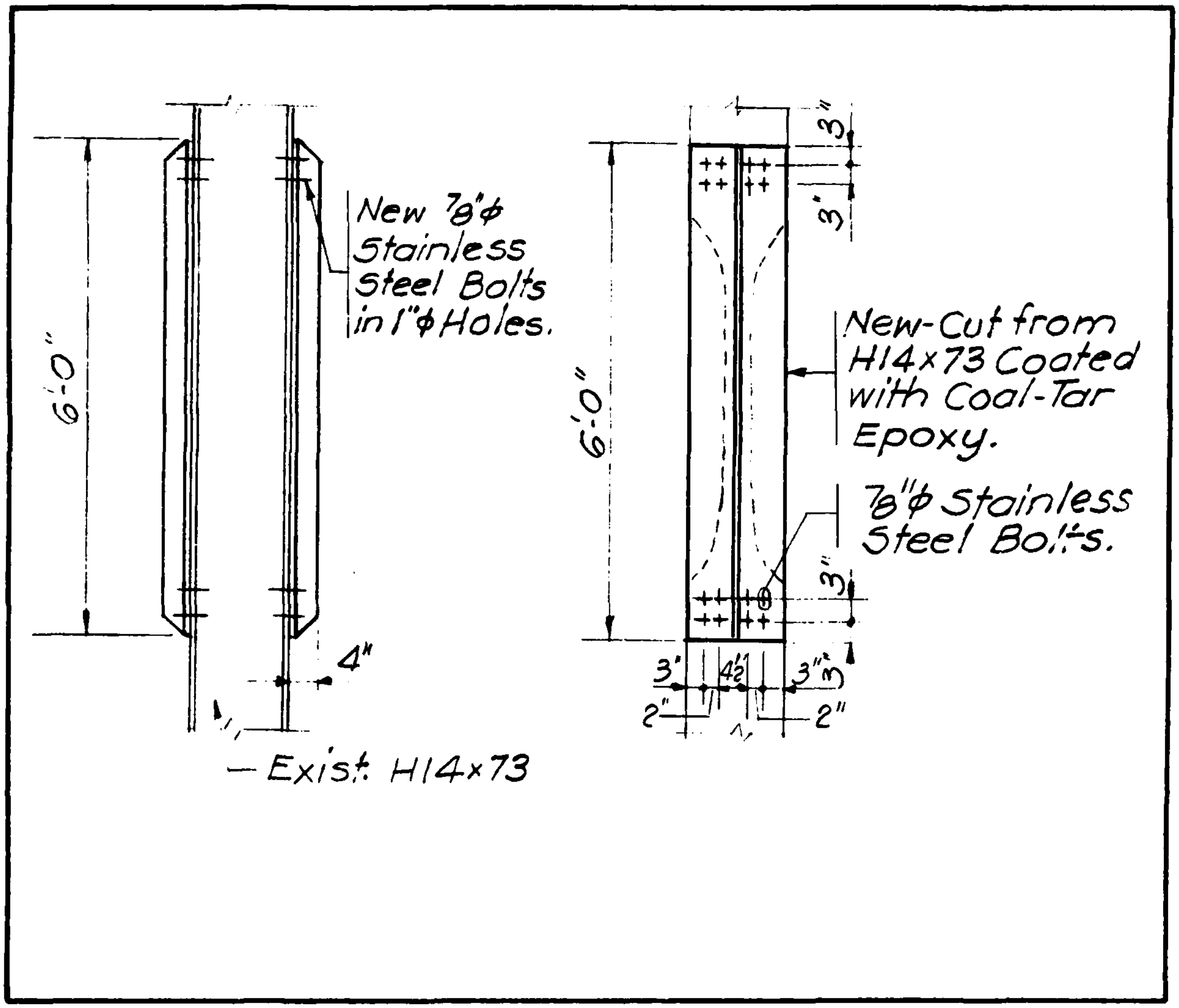


FIGURE 56

Case History No. 4 -
Repair Details

FIGURE 57
Case History No. 5 -
Deterioration of Timber Pile

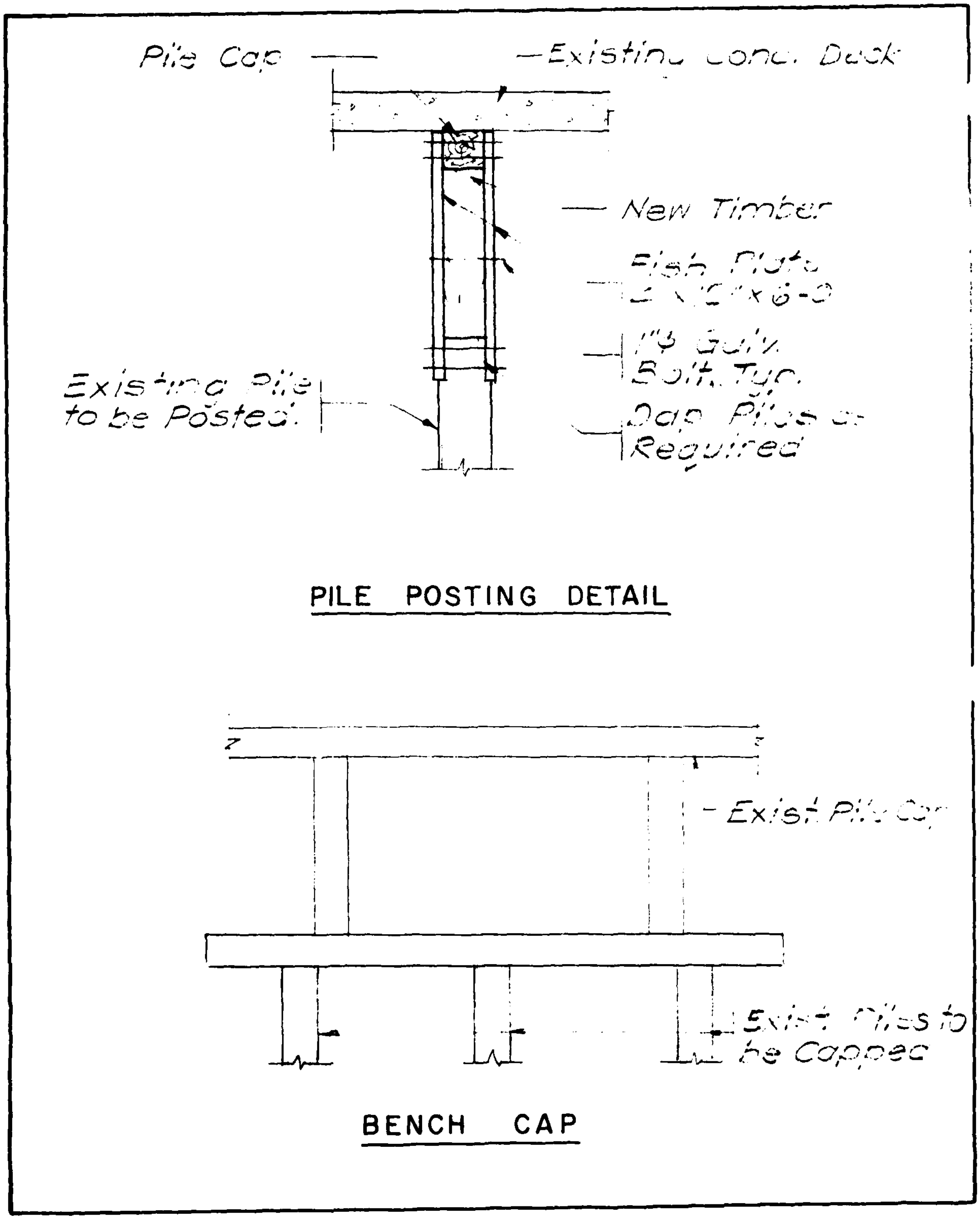
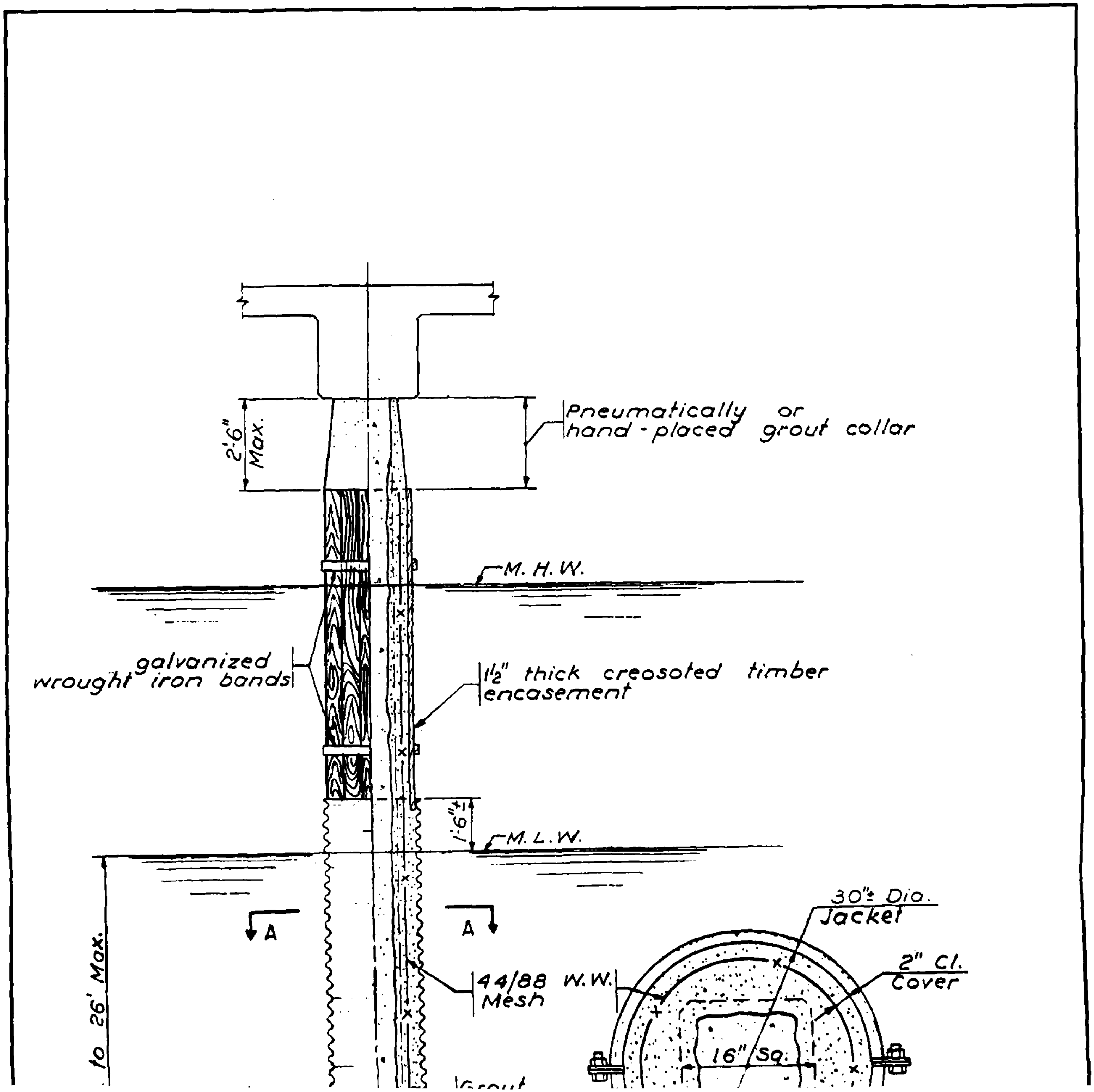


FIGURE 58
Case History No. 5 -
Repair Details



Pneumatically or hand-placed grout collar
2'-6" Max.
M.H.W.
galvanized wrought iron bands
1½" thick creosoted timber encasement
1'-6"±
M.L.W.
A
A
to 26' Max.
44/88 W.W. Mesh
30"± Dia. Jacket
2" Cl. Cover
16" Sq.

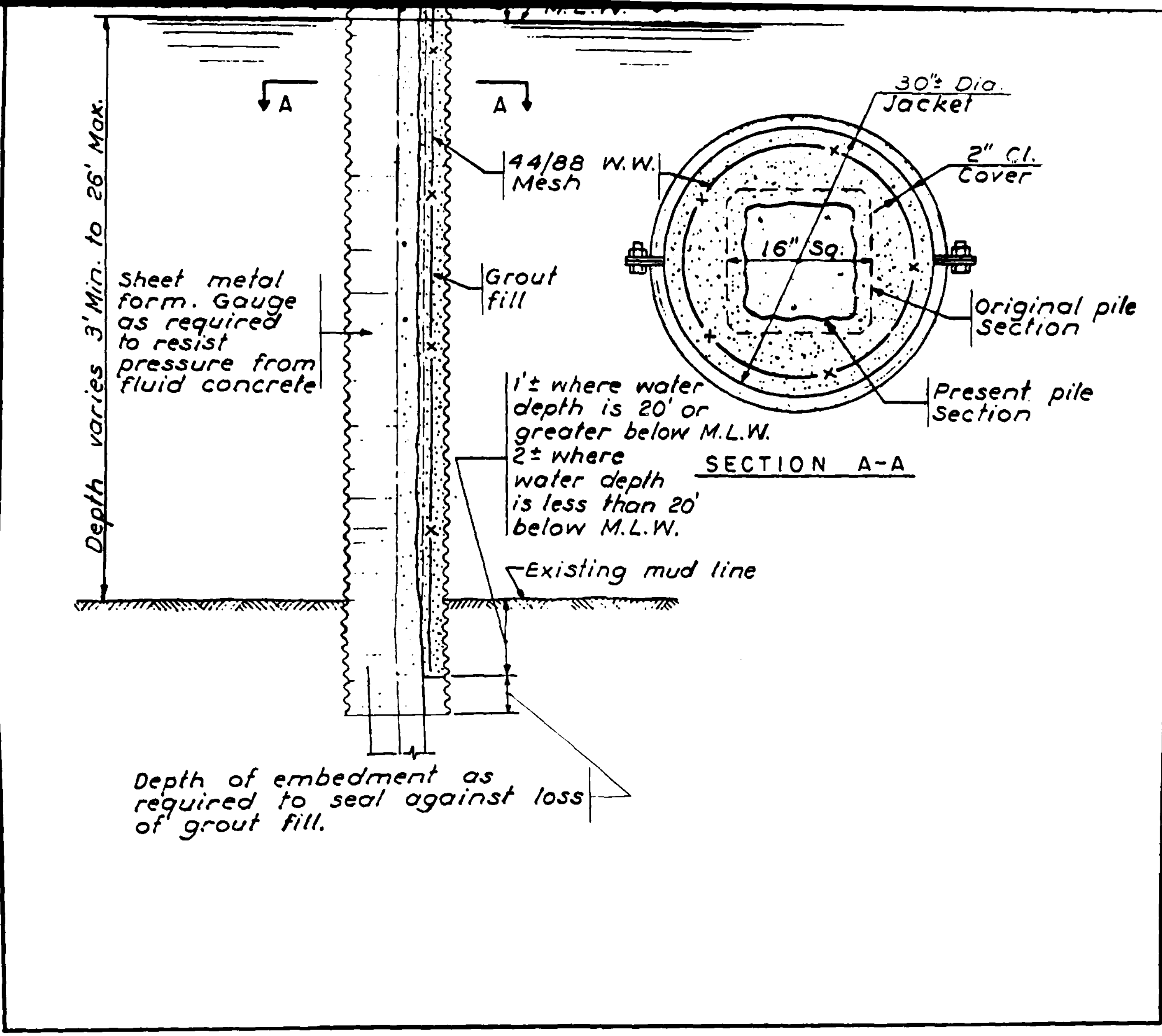


FIGURE 59

Case History No. 6
Repair Details

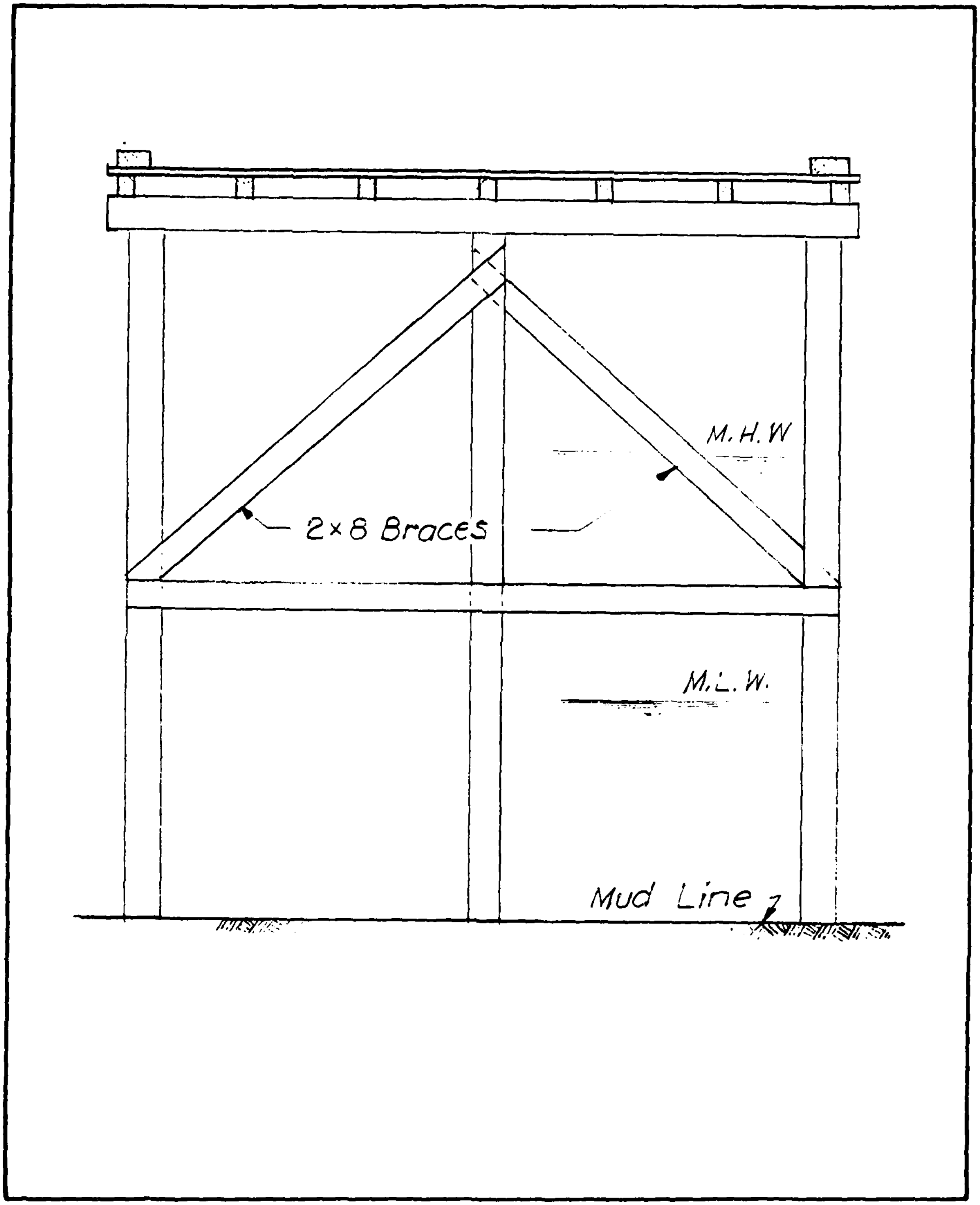


FIGURE 60
Case History No. 7 -
Cross Section of Existing Pier
25.6-97

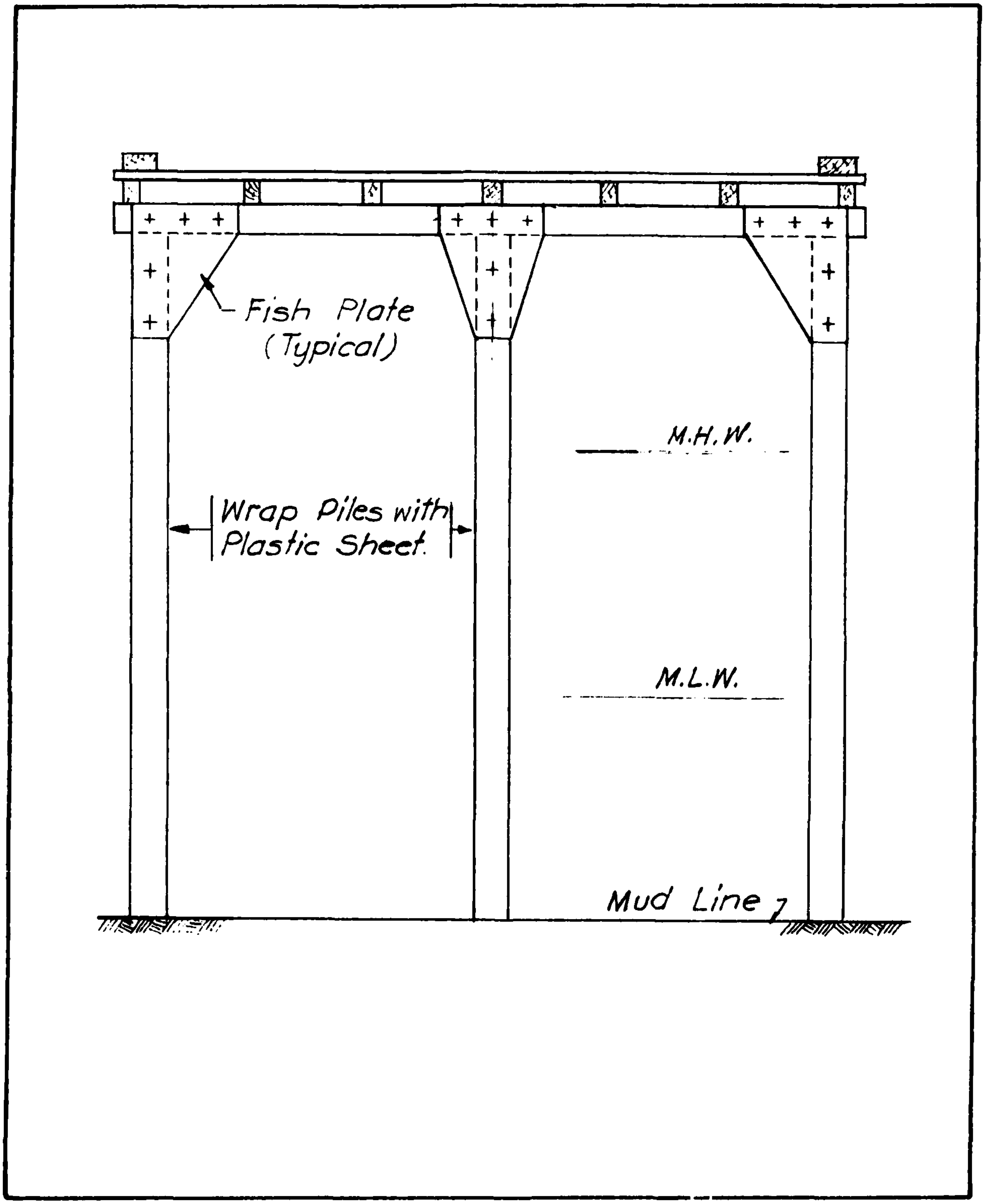


FIGURE 61
Case History No. 7 -
Repair Details

b. The Solution. Do nothing! (judicious neglect). Profile of residual thickness of metal shown in Figure 5. Calculations show enough column capacity remaining to provide substantial service life before critical conditions would develop. Economic analysis in accordance with P-442 showed overwhelming advantage to deferring repair. With high discounting rates (8 percent or more), this is the usual conclusion.

CASE HISTORY NO. 9

a. The Problem. Corrosion of steel sheet piling of cell structure and of steel framing supporting the fender system. (See Figures 62 and 63.) Corrosion profile for steel sheet piling is shown in Figure 4.

b. The Solution. Reinforce steel framing by plating as shown in Figure 64. Only webs had substantially thinned. Flanges were free draining and had lost negligible section. Jacket cells in active corrosion zone using detail shown in Figure 65. Active corrosion zone was of limited height--at or near mean low water.

CASE HISTORY NO. 10

a. The Problem. Corrosion of steel bracing for piling. (See Figure 66.)

b. The Solution. Remove the braces. The braces were not replaced. Careful analysis indicated that they were not a necessary adjunct if restraint of trestle structure due to limited movement which could develop before deck butted against cells and approach abutment were considered.

FIGURE 62
Case History No. 9 -
Corrosion of Steel Sheet Piling
25.6-100

FIGURE 63
Case History No. 9 -
Corrosion of Steel Framing

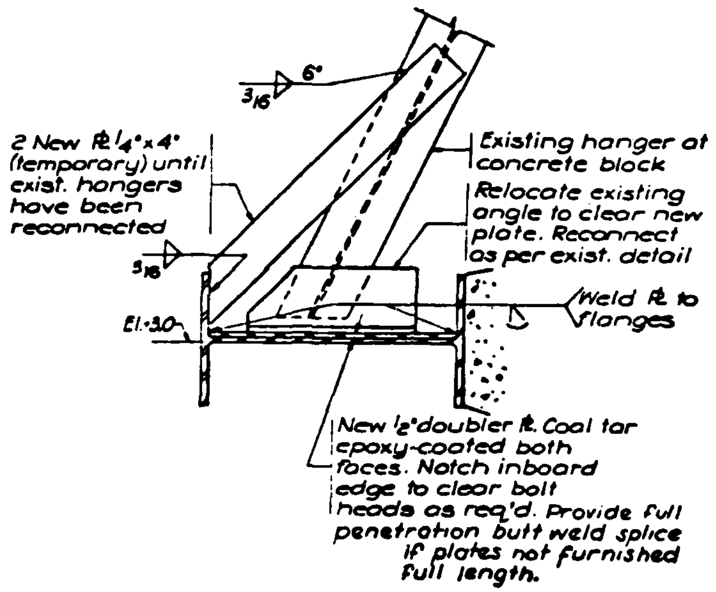
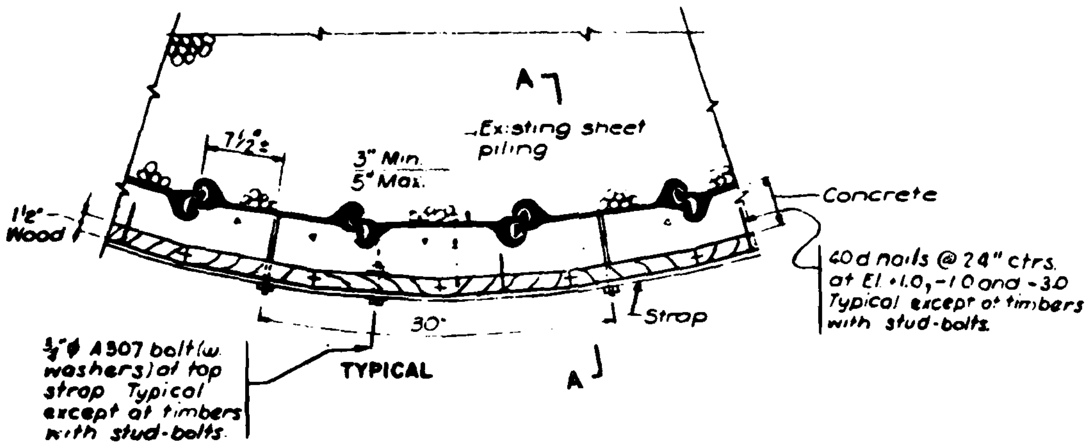
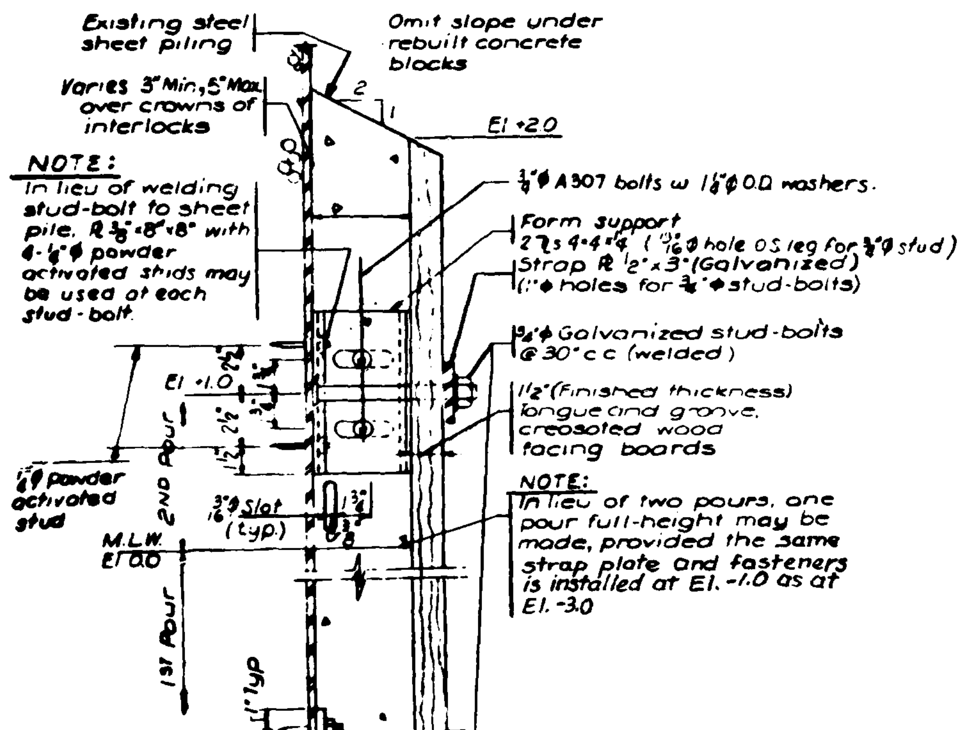


FIGURE 64
Case History No. 9 -
Repair Details - Steel Framing



A
Existing sheet piling
$7\frac{1}{2}$"±
3" Min.
5" Max.
$1\frac{1}{2}$" Wood
Concrete
40d nails @ 24" ctrs. at El +1.0, -1.0 and -3.0 Typical except at timbers with stud-bolts.
30"
Strop
$\frac{3}{4}$"ϕ A307 bolt (w. washers) at top strap Typical except at timbers with stud-bolts.
TYPICAL
A

PLAN



Existing steel sheet piling
Omit slope under rebuilt concrete blocks
Varies 3" Min, 5" Max over crowns of interlocks
El +2.0
NOTE:
In lieu of welding stud-bolt to sheet pile, ℞ $\frac{3}{8}$"×8"×8" with 4-$\frac{1}{4}$"ϕ powder activated studs may be used at each stud-bolt.
$\frac{3}{4}$"ϕ A307 bolts w $1\frac{1}{8}$"ϕ O.D washers.
Form support
2 ⌐s 4×4×$\frac{1}{4}$ ($\frac{13}{16}$"ϕ hole OS leg for $\frac{3}{4}$"ϕ stud)
Strap ℞ $\frac{1}{2}$"×3" (Galvanized)
(1"ϕ holes for $\frac{3}{4}$"ϕ stud-bolts)
$\frac{3}{4}$"ϕ Galvanized stud-bolts @ 30" c c (welded)
El +1.0
$1\frac{1}{2}$" (Finished thickness) tongue and groove, creosoted wood facing boards
$\frac{1}{4}$"ϕ powder activated stud
2ND Pour
$\frac{3}{16}$"ϕ Slot (typ.)
NOTE:
In lieu of two pours, one pour full-height may be made, provided the same strap plate and fasteners is installed at El. -1.0 as at El. -3.0
M.L.W. El 0.0
1ST Pour
1" Typ

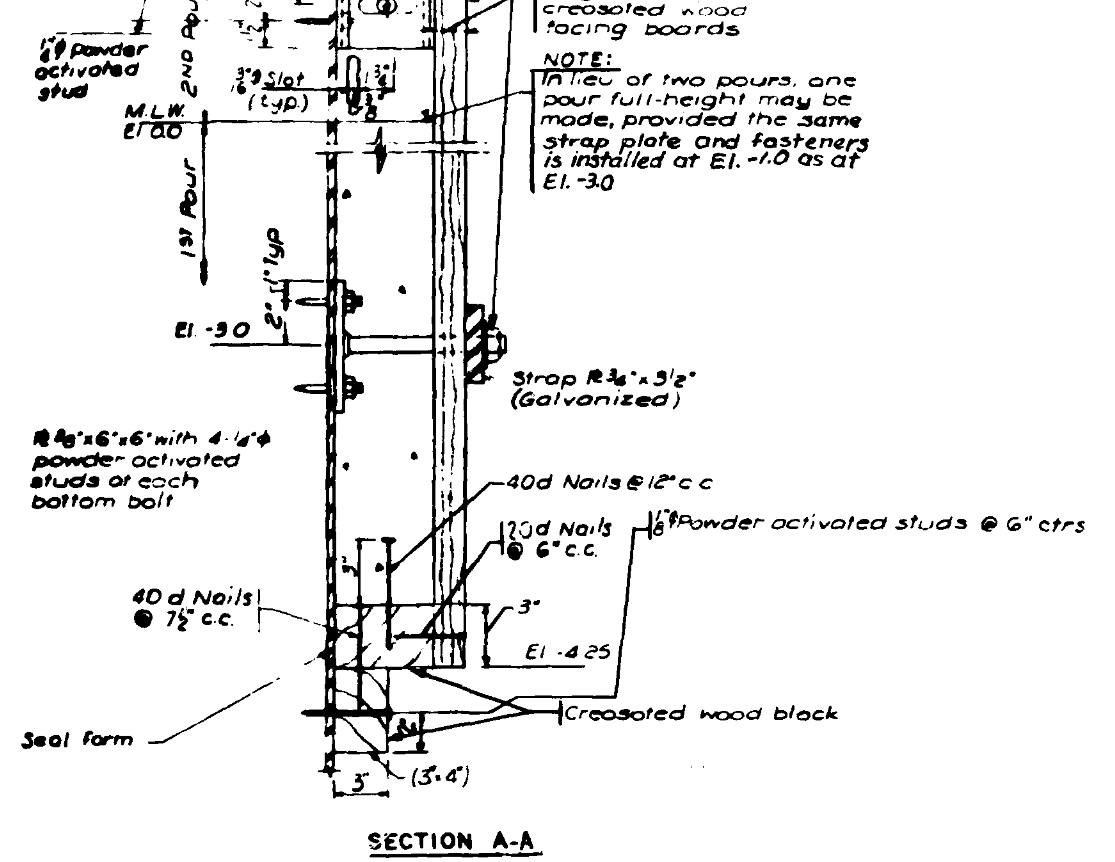


FIGURE 65
Case History No. 9 -
Repair Details - Steel Sheet Piling

FIGURE 66
Case History No. 10 -
Corrosion of Steel Bracing

REFERENCES

NAVFACENGCOM Design Manuals and P-Publications

DM-2.1	Structural Engineering: General Requirements
DM-2.3	Structural Engineering: Steel Structures
DM-2.4	Structural Engineering: Concrete Structures
DM-2.5	Structural Engineering: Timber Structures
DM-7	Soil Mechanics, Foundations, and Earth Structures
DM-8	Fire Protection Engineering
DM-25.1	Piers and Wharves
DM-25.2	Dockside Utilities for Ship Service
DM-25.4	Seawalls, Bulkheads, and Quaywalls
DM-26.1	Harbors
DM-26.6	Mooring Design, Physical and Empirical Data
P-442	Economic Analysis Handbook

Government agencies may obtain Design Manuals and P-Publications from the U.S. Naval Publications and Forms Center, 5801 Tabor Ave., Philadephia, PA. 19120. TWX: 710-670-1685, AUTOVON: 442-3321. The stock number is necessary for ordering these documents and should be requested from the NAVFACENGCOM Division in your area.

NON-Government organizations may obtain Design Manuals and P-Publications from the Superintendent of Documents, U.S. Government Printing Office, Washington, DC 20402.

ACI Standard 318, Building Code Requirements for Reinforced Concrete, American Concrete Institute, Box 19150, Redford Station, Detroit, Michigan 48219.

American Society for Testing and Materials (ASTM), 1916 Race Street, Philadelphia, PA 19103.

A6	Standard Specification for General Requirement for Rolled Steel Plates, Shapes, Sheet Piling and Bars for Structural Use
A36	Specification for Structural Steel
A242	Specification for High-Strength, Low-Alloy Structural Steel
A252	Welded and Seamless Steel Pipe Piles
A588	Specification for High-Strength, Low-Alloy Structure Steel with 50,000 psi Minimum Yield Point to 4 in. Thick
A690	The Specification for High-Strength, Low-Alloy Steel H Piles and Sheet Piling for Use in Marine Environments
A709	Specification for Structural Steel for Bridges

C42 Obtaining and Testing Drilled Cores and Sawed Beams of Concrete

D25 Specification for Round Timber Piles

D245 Standard Methods for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber

AWPB Standards, American Wood Preservers Bureau, P.O. Box 6085, 2772 S. Randolph Street, Arlington, VA 22206.

MP-1 Standard for Dual Treatment of Marine Piling Pressure Treated with Water-Borne Preservatives and Creosote for Use in Marine Waters

MP-2 Standard for Marine Piling Pressure Treated with Creosote for Use in Marine Waters

MP-4 Standard for Marine Piling Pressure Treated with Water-Borne Preservatives for use in Marine Waters

MLP Standard for Softwood Lumber, Timber and Plywood Pressure Treated for Marine (Saltwater) Exposure

Construction and Protection of Piers and Wharves, NFPA No. 87, National Fire Protection Association, 470 Atlantic Avenue, Boston, MA 02210.

OPNAVINST 5510.45B, U.S. Navy Physical Security Manual.

NAVFAC P-272 - Definitive Designs for Naval Shore Facilities

Drawing No.	Title
1293323	Berthing Pier - Types of Construction - Sheet No. 2

NAVFACENGCOM Guide Specifications (TS Series)

TS-02310	Round Timber Piles
TS-02312	Prestressed Concrete Piling
TS-02315	Steel H-Piles
TS-02317	Cast-in-Place Concrete Piles, Steel Casing
TS-02318	Auger Placed Grout Piles
TS-02319	Round Timber - Concrete Composite Pile

NAVFAC Guide Specifications are available, free of charge, from the U.S. Naval Publications and Forms Center, 5801 Tabor Ave., Philadelphia, PA. 19120. TWX: 710-670-1685, AUTOVON: 442-3321.

GLOSSARY

Apron. Clear area around perimeter of a dock for parking, work area, access, and storage.

Bent. Transverse pile framing in the substructure of a pier or wharf.

Berthing Basin. Area of harbor set aside for berthing vessels at docks and/or open anchorages.

Bottom. The ground or bed under any body of water; the bottom of the sea.

Breaking Out. Setting up; preparing.

Bulkhead. See Section 2, paragraph 1.b. of DM-25.4.

Bulkhead Lines. Lines which establish limits outside of which continuous solid-fill construction is not permitted.

Caisson. (1) A watertight box used to surround the works involved in laying the foundation of a bridge or other structure below water.

(2) A watertight box used as a closure for graving dock entrances.

Camel. Floats placed between vessel and dock, or between vessels, designed to distribute wind and current forces acting on the vessel.

Chock. (1) A horizontal component of a fender system used to brace the vertical piles or fenders.

(2) A mooring fitting having curved ends for guiding lines.

Cofferdam. A temporary wall serving to exclude water from any site normally under water so as to facilitate the laying of foundations or other similar work.

Controlling Depth. The least depth in the navigable parts of a waterway, governing the maximum draft of vessels that can enter.

Cover. The thickness of concrete between the outer surface of any reinforcement and the nearest surface of the concrete.

Current. A flow of water.

Dap. Notches in timber to provide flat bearing surface.

Deadman. Concrete, plate, or other anchorage for a land or water tie.

Diaphragm. (1) Short transverse member connecting to longitudinal stringers.

(2) Transverse piling in sheet-pile cofferdam.

Dock. A pier or wharf used for berthing vessels and for transfer of cargo or passengers.

Dolphin. A structure usually consisting of a cluster of piles. It is placed near piers and wharves or similar structures, or alongshore, to guide vessels into their moorings, or to fend vessels away from structures, shoals, or the shore.

Draft. Depth of vessel hull below the waterline.

Dredge Line. Line establishing limit of dredging.

Ebb Tide. The period of tide between high water and the succeeding low water; a falling tide.

Fender. A device or framed system placed against the edge of a dock, to take the impact from berthing or berthed vessel.

Flood Tide. The period of tide between low water and the succeeding high water; a rising tide.

Freeboard. Distance between the weather deck of a floating vessel and the water line.

Harbor. In general, a sheltered arm of the sea, easily accessible to maritime routes in which ships may seek refuge, transfer cargo, and/or undergo repair.

Harbor Lines. Lines which control the location of shore structures in or adjacent to navigable waters.

Hurricane. An intense tropical cyclone in which winds tend to spiral inward toward a core of low pressure, with maximum surface wind velocities that equal or exceed 80 mph (70 knots) for several minutes or longer at some points. Tropical Storm is the term applied if maximum winds are less than 80 mph.

Lagging. Horizontal timber sheeting commonly used in bulkhead walls.

Lee. (1) Shelter, or the part or side sheltered or turned away from the wind or waves.

(2) The quarter or region toward which the wind blows (chiefly nautical).

Mean High Water (MHW). The average height of the high water over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value.

Mean Low Water (MLW). The average height of the low waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value.

Mean Sea Level. The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings.

Mole. A massive land-connected, solid fill structure of earth (generally revetted) masonry, or large stone. It may serve as a breakwater or pier.

Pier. A dock that is built from the shore out into the harbor and used for berthing and mooring vessels.

Pierhead Lines. Line which establish the outboard limit for open pier construction.

Quarry Run Stone. Stone as it is excavated from the quarry with no screening.

Quaywall. See Section 2, paragraph 1.c. of DM-25.4.

Relieving Platform. A platform supported by piles, employed in wharf construction, to relieve lateral pressures from surcharge.

Revetment. A facing of stone, concrete, or other form of armor built to protect a scarp, embankment, or shore structure against erosion by wave action or current.

Rigid Frame. A rigid joint structure in which moments and shears in joints maintain the equilibrium of the structure.

Riprap. A layer, facing, or protective mound on stones randomly placed to prevent erosion, scour, or sloughing of a structure or embankment; also the stone so used.

Scuppers. Openings for drainage of water off a pier, wharf, or bridge deck.

Seawall. See Section 2, paragraph 1.a. of DM-25.4.

Shoreline. The intersection of a specified plane of water with the shore or beach (e.g., the highwater shoreline would be the intersection of the plane of mean high water with the shore or beach). The line delineating the shoreline on National Oceanic and Atmospheric Administration nautical charts and surveys approximates the mean high water line.

Significant Wave Height. The average height of the one-third highest waves of a given wave group. Note that the composition of the highest waves depends upon the extent to which the lower waves are considered. In wave record analysis, the average height of the highest one-third of a selected number of waves, this number being determined by dividing the time of record by the significant period.

Slip. A space between two piers for berthing a vessel.

Soldier Beam. Vertical beam used to resist lateral pressure through cantilever action.

Splash Zone. The range between mean low water and the upper limit to which the structure could be expected to be wetted by average wave disturbance at the site.

Sponson. Overhanging section of vessel deck.

Stringer. A longitudinal member in a structural framework.

Swell. Wing-generated waves that have traveled out of their generating area. Swell characteristically exhibits a more regular and long period, and has flatter crests than waves within their fetch.

Tidal Prism. The total amount of water that flows into a harbor or estuary or out again with movement of the tide, excluding any freshwater flow.

Tidal Range. The difference in height between consecutive high and low waters.

Tide. The periodic rising and falling of the water that results from gravitational attraction of the moon and sun and other astronomical bodies acting upon the rotating earth.

Wale. A horizontal component of a fender system generally placed between the vertical fenders and the pier structure and used for horizontal distribution of forces from a vessel.

Waterline. A juncture of land and sea. This line migrates, changing with the tide or other fluctuation in the water level.

Wave. A ridge, deformation, or undulation of the surface of a liquid.

Wave Height. The vertical distance between a crest and the preceeding trough. (See also significant wave height).

Wharf. A dock, oriented approximately parallel to shore and used for berthing or mooring vessels.

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